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Abatement of Deposition and Scour in Sewers

Water Resources Engineers, Inc, Walnut Creek, Calif

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Feasible methods are identified for reducing first-flush pollution in new and existing storm and combined sewer systems. A mathematical model is described which was developed to simulate the behavior of solids in pipelines and to evaluate the costs of first-flush abatement alternatives.

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FOREWORD

The Environmental Protection Agency was created because of increasing public and government concern about the dangers of pollution to the health and welfare of the American people. Noxious air, foul water, and spoiled land are tragic testimony to the deterioration of our natural environment. The complexity of that environment and the interplay between its components require a concentrated and integrated attack on the problem.

Research and development is that necessary first step in problem solution and it involves defining the problem, measuring its impact, and searching for solutions. The Municipal Environmental Research Laboratory develops new and improved technology and systems for the prevention, treatment, and management of wastewater and solid and hazardous waste pollutant discharges from municipal and community sources, for the preservation and treatment of public drinking water supplies, and to minimize the adverse economic, social, health, and aesthetic effects of pollution. This publication is one of the products of that research; a most vital communications link between the researcher and the user community.

This report describes new methodology for analyzing the design of storm and combined sewer systems. Particularly the work addresses the abatement of pollution caused by the flushing of accumulated solids from sewers during storm periods.

Francis T. Mayo
Director
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ABSTRACT

The research reported here had two major objectives: a) a review of feasible methods for reducing the first-flush of solids in overflows from storm and combined sewers, and b) the development of a mathematical model that could be used to evaluate the behavior and cost of first-flush mitigating alternatives.

Feasible methods of controlling first-flush pollution in new designs include increasing conduit slopes, providing cunette sections or other pipe shapes that maintain high velocity during low flow periods, providing upstream capture of solids during high flows, and providing increased amounts and degrees of treatment downstream. Flushing of sewers already in place appears to be the most feasible method of reducing first-flush pollution in existing systems. All these alternatives are directed toward minimizing accumulation of solids in pipelines and toward providing the maximum amount of treatment that proves cost effective.

A model was developed, adapted from the existing EPA Storm Water Management Model, which simulates the behavior of solids in pipelines as bedload, suspended load, or washload. Example problems are described which demonstrate both the workings of the model and the improvements in first-flush mitigation that the model can identify.

This report was submitted in fulfillment of Contract No. 68-03-2205 by Water Resources Engineers, Inc. under the sponsorship of the U.S. Environmental Protection Agency. This report covers work performed during the period June 1, 1975 to October 31, 1976, and work was completed as of November 30, 1976.

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SECTION I

CONCLUSIONS

Deposition and scour of solid materials in sewer systems have been studied to evaluate alternative methods for alleviating these problems, and to demonstrate how deposition and scour and their abatement can be evaluated through mathematical modeling.

Phase I of the study listed available methods for alleviating the flushing of deposited solids into receiving waters during high-flow, storm-runoff periods when treatment plants designed for smaller flows are not able to treat the entire storm volume. Moreover, Phase I determined the feasibility of mathematically simulating the improvements afforded by these alternatives. Phase II, which is the major topic of this final report, included modeling of the deposition, scour and transport processes and estimating the costs of the simulated facilities. From both phases of the work, but especially from Phase II, the following conclusions have been reached.

1) Sewer design based on a velocity criterion only for self-cleansing does not consider all the relevant variables that affect deposition and scour, and hence such design can lead to situations in which deposition may still occur and "first-flush" pollution will not be prevented. Where velocity becomes the controlling factor, however, 2 feet per second appears to be adequate to initiate scour of deposited materials. This conclusion is based only on the analyses used here which assume noncohesive deposits are involved.

2) Sediment transport theory derived over the years for natural streams, steady-state conditions, and beds having inexhaustible supplies of particles can be modified and used adequately to evaluate solids transport problems in man-made pipe systems operating under transient conditions and having finite amounts of deposited materials.

3) A combined sewer system configuration can be found that will have about equal cost and solids capture efficiency as a comparably designed separate storm and sanitary system.

4) Analyses of example cases presented later in this report led to some encouraging but unexpected results. For example, one case included swirl concentrators at upstream points in

a combined sewer system and secondary treatment at the downstream end of the system. Another case included the same basic features but larger capacities of the swirl concentrators and secondary treatment and the addition of sand filtration to the secondary plant. Additionally, the second case had pipes one foot smaller in diameter than in the first case and laid at a somewhat higher slope. The savings in the second example in pipe costs and excavation costs (topographic control was at the downstream end) more than offset the increased treatment costs; while the solids capture efficiency increased from about 50 to over 80 percent. In this case the "better" system was the cheaper one. In other cases, solids capture efficiencies were more than doubled by adding pipe and/or treatment facilities costing only 5 to 10 percent more.

Admittedly, these precise findings are related only to the physical dimensions of the example problems analyzed here. But the findings are significant because they are not likely to be predicted. The general conclusion to be drawn is that design of systems of connected pipes, storage facilities, and treatment devices can and should include analyses of a wide variety of options, each of which will have its peculiar physical and economic consequences. The design of the total system should be the objective, not the individual design of its parts; and fortunately a model now exists to assist that process.

SECTION II

RECOMMENDATIONS

It is recommended that the new updated sewer analysis model reported here be applied in a variety of field applications, both to test its relationships and inherent assumptions, and to allow sewer cleansing evaluations and designs to be performed with regard given to a number of the contributing variables in addition to velocity in the pipes.

Considerably more research should be undertaken in the area of sediment transport in pipelines. While the model developed here works, in the sense that it functions and yields helpful answers, it is by no means perfect or perfectly general. Particularly important topics for research are the definitions of and separations among bedload, suspended load, and washload; extension of the relationships now available to account for stickiness and armoring of deposited materials; and the linkages between "suspended solids" and other constituents such as organic matter, heavy metals, and nutrients often reputed to be associated with solids.

The claimed superiority for pollution control of separate storm and sanitary sewers over combined sewers should be reevaluated as a concept in light of the preliminary findings here that combined sewer systems can be comparable in cost but superior in solids capture efficiency to separate systems.

SECTION III

SCOPE, OBJECTIVES, AND SUMMARY OF FINDINGS

PHASE I

The initial phase of this project investigated a) the available alternative methods for reducing first-flush pollution, and b) the feasibility of simulating these methods for its reduction. During this phase the following tasks were performed:

1) First-flush control alternatives were identified.

2) Cost-size relationships for the identified alternatives were found in existing literature.

3) Solids removal efficiencies of the alternative methods were obtained from literature reviews.

4) The feasibilities of a) using these alternatives in the field and b) simulating their behavior and estimating their costs in a mathematical model were determined.

The findings at the end of Phase I were somewhat paradoxical. Some of the alternative control measures appearing most feasible for field application, particularly in existing sewer systems, were going to be the most difficult to simulate. The converse was also true: the things that could be changed most easily in a model, such as pipe diameter or slope, were the least feasible in the field, especially in existing sewer systems.

Nonetheless, it was concluded that a number of feasible modeling alterations to include first-flush mitigation measures could and should be made to an existing sewer analysis model, namely the Storm Water Management Model (SWMM), which EPA had developed and promulgated over a number of years. Thus Phase II was then directed to making these changes.

Specific and more detailed findings from Phase I are summarized below.

1) Three major categories of first-flush control alternatives for sewer systems have been tried in field applications, at least as demonstration projects. These include a) pipe configuration changes of size, slope, and shape; b) modifications at sewer inlets such as inflatable dams to store some dry weather flow in an upstream pipe, later to be released as a flushing wave into downstream sewers; and c) upstream and

downstream storage and treatment alternatives, including detention basins, swirl concentrators, and conventional waste treatment with primary and secondary plants.

2) Alteration of pipe configurations to control or lessen first-flush pollution in existing sewer systems can be very difficult or even impossible. For example, if it were found that a pipe in the middle of a sewer network could be kept clean during dry weather by tripling its slope, such an alteration could prove to be hydraulically infeasible and certainly not cost effective. In the design of new systems, however, consideration of self-cleansing properties of sewers has been a standard and fairly simple practice. One attempts to find a pipe diameter and slope combination that yields a minimum velocity at half-full or full flow of 2 feet per second, and a condition of at least half-full flow once a day. Where slope and other conditions prevail against a combination of these two variables that is cost effective, a change in pipe shape to egg-shaped or cunette cross-section is considered, whereby low flows will move with velocities and hence scour potentials rivaling those at half-full or full flow.

A happenstance that occurs, regardless of design considerations, is that excessive deposition does occur in some pipes of existing sewer systems. When deposition becomes excessive, sometimes to the extent that it causes loss of hydraulic capacity, the alternatives other than changing the pipe configuration are to flush the sewer or to clean it by hand. In San Francisco there was a case, though, where deposition was so rapid and so excessive and the deposits became so encrusted that continuous cleaning by hand or flushing at upstream manholes were not as feasible as changing the pipe. The old sewer section was removed and replaced with one having a precast cunette section in the bottom, which allowed the solids to be continually moved toward a downstream treatment plant. While this action was taken to prevent upstream flooding caused by the clogging of the sewer, and not for the control of first-flush pollution; the exception to the rule of infeasibility of changing existing pipes to solve deposition problems is of interest.

With respect to the feasibility of simulation of these alternatives, changing size and slope in a model like SWMM is absolutely trivial. One simply has to retype one number on one card and make a new simulation. Changes in conduit shape are a bit more difficult to treat, although the SWMM version ultimately adapted in this project already treated egg-shaped, baskethandle, horseshoe, rectangular, and trapezoidal sewer sections, in addition to circular sewers. Cunette sections were not originally included, and this required some reprogramming in Phase II.

3) Modifications at sewer inlets, including flushing with piped-in freshwater or with temporarily stored upstream dry weather flows, capture of solids in catchbasins or offline storage manholes, or capture of solids from street runoff in filter bags hung below grate inlets have all been tried. A variety of operating problems have been reported. Of these, flushing with freshwater introduced for this purpose appears to be the most feasible, and research is going forward currently to determine operating methods and efficiencies (Pisano, 1976).

   From a simulation point of view, it was decided that flushing alone was the only one of these methods currently feasible and sufficiently proved in practice to warrant introduction to the model. Since flushing is practiced by sending a wave of water through the sewer, having precisely the same characteristics as a storm hydrograph which the model already handled, no modifications to the model were necessary to evaluate flushing.

4) Storage and treatment alternatives have obvious and compelling logic as first-flush control methods: to stop polluting overflows, store the excess water for later treatment. The stormwater detention cases studied from the literature, however, all were plagued with the difficulty or merely lack of consideration of removing the solids after the storm had passed from the basin, lagoon, rubber bag, or whatever the storage device may have been. Because that fairly essential part of the process had not been addressed and characterized in the literature, it was decided not to attempt addition of storage and solids capture capability to the model.

   On the other hand, treatment devices such as swirl concentrators for upstream solids capture during storm flows and the conventional downstream methods of waste treatment have proved so effective in removal of solids that it was deemed essential to add capability to the model for simulating their quality improvements. Because the physics and chemistry of behavior of these devices is still not well understood, actual representation of their behavior was not attempted in the model. But fixed removal percentages were estimated for each kind of treatment device, and their costs were estimated as well.

PHASE II

The following tasks were undertaken during Phase II.

1) Add simulation capability to an existing sewer transport model. Modifications were to include simulation of the deposition and scour phenomena, as well as the influence of feasible first-flush control alternatives.

2) Add cost estimation capability to the model. The decision was made during the project to add this capability to the existing model, rather than to develop a new, separate program.

3) Demonstrate the added model capabilities. At least five simulations were to be made. Both behavioral differences in the physics of solids transport and differences in cost were to be demonstrated for alternative pipe and plant configurations.

The Model to Be Adapted

The remainder of this report deals with the changes that were made to the existing model of sewer behavior. It is appropriate to introduce that model at the outset. It is described in greater detail later in this report.

The EPA SWMM model was developed in 1971 by Metcalf & Eddy, Inc., the University of Florida, and Water Resources Engineers, Inc. (Metcalf & Eddy, *et al*., 1971). It was comprised of three parts or Blocks: a) the Runoff Block, which converted rainfall on urban watersheds to runoff and computed hydrographs and pollutant mass loadings at sewer inlets; b) the Transport Block which routed the hydrographs and pollutant masses through the sewer system; and c) the Receiving Water Block, which simulated the effects on rivers and estuaries of the storm and sanitary inflows provided by the Transport Block.

Since that time, a different version of the Transport Block has been provided to EPA for inclusion in the SWMM package (Shubinski and Roesner, 1973). This option is known as the WRE-SWMM Transport Block. It was this set of programs that was adapted to include deposition and scour and new cost estimation capability during this project.

The WRE-SWMM Transport Block consists of two computer programs. One is the Hydraulics Model, which estimates dry weather flow and quality inputs, accepts hydrograph inputs at specified junctions in the sewer network, and routes the water downstream. The second program is the Quality Model which accepts as inputs the computed pipe flows, junction heads and dry weather mass inputs from the Hydraulics Model, accepts either from cards or from a Runoff Block output tape the mass inputs from storm inflows at junctions, and computes the mass balance of BOD, coliforms, and "suspended solids" throughout the pipe system for the period of simulation. Suspended solids, like the other constituents, are represented as a conservative constituent.

In this project, minor changes were made to the Hydraulics Model, but some substantial additions were made to the Quality Model, all of which are described in detail later in this report.

Chronology and Results of Phase II

In the sequence reported below, the various model modifications were made and the two new programs were tested with example problems devised for this purpose.

1) The literature was studied at great length to glean equations for representing the phenomena of deposition, scour, and solids transport.

2) A deposition and solids transport subroutine was written and incorporated in the Quality Model. This subroutine calculates the movement of solids from one junction to another and the accumulation of solids on pipe bottoms, as appropriate for the hydraulic conditions simulated in each time step.

3) A special subroutine was written and added to the Quality Model to compute the settling velocity of any solid particle of a given diameter and specific weight. The subroutine is used as necessary to compute quiescent water settling velocities for up to 10 different particle size ranges, which are routed through the sewer system.

4) Two other subroutines were developed and added to the Quality Model. The first of these summarizes and prints the results of the sediment transport computations for each pipe at specified time intervals. The second subroutine summarizes for each treatment plant in the simulated system the amount of solids removed during the entire simulation period and prints information about the plant's size and type.

5) Alterations were made in several subroutines of the existing Hydraulics and Quality Models to incorporate conduits with cunette sections. In addition to the six conduit shapes listed previously, both models can now treat conduits with the following shapes: circular with semicircular cunette, circular with rectangular cunette, and rectangular with rectangular cunette.

6) Cost equations for construction and operation of pipe systems and treatment plants were derived from available literature. Pipe costs are now computed and reported by the Hydraulics Model. Treatment costs are computed and reported by the Quality Model.

7) An example combined sewer system was devised consisting of five junctions and four pipes, with a treatment device of any size and type specified at the end of any pipe. An example separate sewer system was also devised, consisting of four pipes and five junctions in both a separate storm sewer system and a separate sanitary system, again with treatment being specified as desired at the end of any pipe section.

8) Approximately 25 simulations were made to show the workings of the new model features and to show the behavior of solid

materials in pipe systems under different conditions of flow and pipe and plant configuration.

The remaining chapters of this report describe the development, testing, and results of these new model features.

SECTION IV

ADAPTATION OF SEDIMENT TRANSPORT THEORY

The literature on sedimentation, sediment transport, channel erosion, and similar or equivalent phenomena is staggering by its sheer volume. Some 25 to 50 books and articles have been reviewed in this project, most of which were written after 1971, and virtually all of which address sediment motion in natural streams rather than pipe systems. All of them will not be referenced here, primarily because in the end all the relationships used in this work were reported in two or three volumes that summarized the state of the art as well. Nonetheless, following the references given at the end of the report, there is a bibliography that includes the other works reviewed. Most of what follows was taken from the book by Graf (1971) on sediment transport and the book by an ASCE Task Committee (1975) on sedimentation engineering.

DEFINITIONS

Three basic loads of solid materials transported by water in a stream or conduit have been defined. These are *bedload*, *suspended load*, and *washload*. Graf (1971) has introduced other terms as well; for example, *bed material load* is represented by the sum of bedload and suspended load. Moreover, he pointed out that the bed material load is often called the *total load*, which is misleading, since more than 50 percent of the total material moving in the channel may be washload.

Bedload consists of particles rolling, sliding, sometimes jumping along close to the channel bottom. Some of the qualifiers or subdefinitions one reads are confusing because this material is considered, even by a single author, to have "deposited", to not remain stationary, or to be derived from material some of which deposits and remains at rest while some of it never does deposit. The most often appearing definition is that bedload is material in motion near the bed. *Suspended load* is comprised of particles that have been swept upward into the flow from the bed. Apparently, they have gone through the experience of being in motion as bedload, at least for an instant; so it can be said that they are derived solely from bedload, and not possibly from the stationary bed directly. The distinction between bedload and suspended load is one of hydrodynamic forces imparted by the flowing water, by gravity, and by the particles' own characteristics. Bedload is simply too large or too heavy to be completely suspended in the water in constant, continuous motion. *Washload*, on the other hand, is material so small or so light that it is carried along with the flowing water virtually as a conservative substance. It may never have been part of the bed, the bedload, or the suspended load. Graf (1971) has concluded in a general way that more than 50 percent of the total material in motion in a natural stream can be in the form of washload, and Rendon-Herrero (1974) has indicated that it can be as much as 95 percent. Graf (1971) also says that washload tends to be derived from materials swept in from the watershed and hence can be quite different from materials in a (natural) streambed. A simplification he reports for estimating the characteristics of washload is that washload can be considered to be materials having diameters less than those in the smallest 10 percent of the bedload, regardless of their source.

ASCE (1975) has given similar, but more definitions for the various components of the total material in motion. For purposes of this report, which deals with quite unnatural channels and very transient conditions, the following definitions will be used.

Bedload is comprised of solid materials that move near the pipe bottom in the direction of flow according to the relationships for such motion given below. Particles of any size may be included in bedload in a given time step. Hence, they may be derived originally from any source, i.e., the watershed, sanitary flow inputs, or the deposits on the pipe bed from the previous time period. However, before being moved as bedload they will have previously deposited to the pipe bottom during a period of sufficiently quiescent flow to permit this, which may be during the same time period in which they are also moved.

Suspended load is comprised of materials that have at one time been deposited and have subsequently been swept from the bedload into the overlying flow with a distribution throughout the depth as described by relationships given below.

Washload is comprised of particles that have just entered the pipe at the upstream junction and have been determined to be unable to settle or deposit (be removed by sedimentation) within the pipe length during the current time period, given the flow conditions then existing. Washload, therefore, is not limited to certain sizes or defined by its original source.

BEDLOAD TRANSPORT RELATIONSHIPS

Of many relationships given by Graf (1971) and ASCE (1975) for bedload transport, the one chosen for use here is that attributed to Kalinske (1947). Of those relationships reviewed, Kalinske's appears to be as reliable as any others, it has been corroborated with data, and it requires values for variables that are knowable from other computations in the Hydraulics and Quality Models used here. It states:

$$\frac{q_s}{u_* d} = f\left[\frac{(\tau_o)_{cr}}{\tau_o}\right] \qquad [1]$$

in which

q_s = volume rate of solids movement per unit width, ft³/sec-ft;

u_* = shear velocity, $\sqrt{\tau_o/\rho}$; ft/sec;

d = median diameter of particles, ft;

τ_o = shear stress = $\gamma\ r_h\ s$, lb/ft²;

$(\tau_o)_{cr}$ = a critical shear stress, obtaining at the moment of incipient motion, lb/ft²;

ρ = density of water, 62.4 lb/ft³ ÷ 32.2 ft/sec² = 1.94 lb-sec²/ft⁴;

γ = specific weight of water, 62.4 lb/ft³;

r_h = hydraulic radius of flow, ft; and

s = the energy slope, ft/ft.

In the model adapted to solve this problem, all these quantities except two are known or can be calculated in each time step for each pipe. The quantity, q_s, is being sought; and a separate expression for $(\tau_o)_{cr}$ is still needed.

The value of the critical shear stress, $(\tau_o)_{cr}$, can be obtained from the relationships of shear stress to particle diameter (in millimeters) shown in Figure 1. This figure was taken from Graf (1971) (Figure 6.6, page 95). The dotted lines indicate the range of data from various investigators as summarized by Graf. The curve for average values fitted for use in this study consists of two lines represented by the equations:

$$(\tau_o)_{cr} = 0.01484\ d^{1.0648} \qquad d \geq 6\ \text{mm} \qquad [2]$$

and

$$(\tau_o)_{cr} = 0.067\ d^{0.2238} \qquad d < 6\ \text{mm} \qquad [3]$$

The units are millimeters for particle diameter and pounds per square foot for critical shear stress. It might be noted that a similar figure in ASCE's book (1975) (Figure 2.44, page 99) shows other data considerably below Graf's range of shear stresses at small diameters. ASCE notes this as well, and is at a loss to explain it, while Graf (1971) states that his curve "summarizes much of the important work done and, hence, should prove very helpful for the hydraulic engineer engaged in channel design." Consequently, Graf's interpretation was accepted for this study.

To solve the relationship in equation 1 for q_s, one more curve is needed. This is given in Figure 2, which is Kalinske's (1947) bedload function.

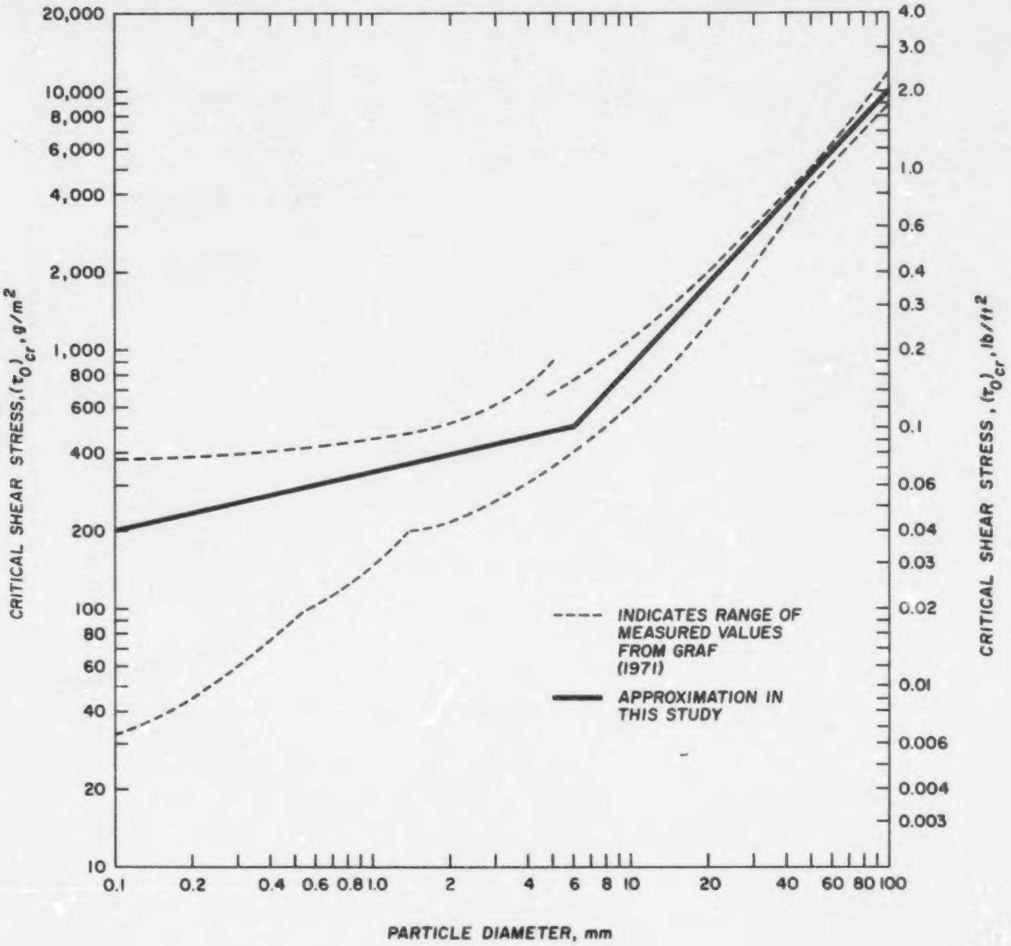


Figure 1
Critical Shear Stress as a Function of Particle Diameter

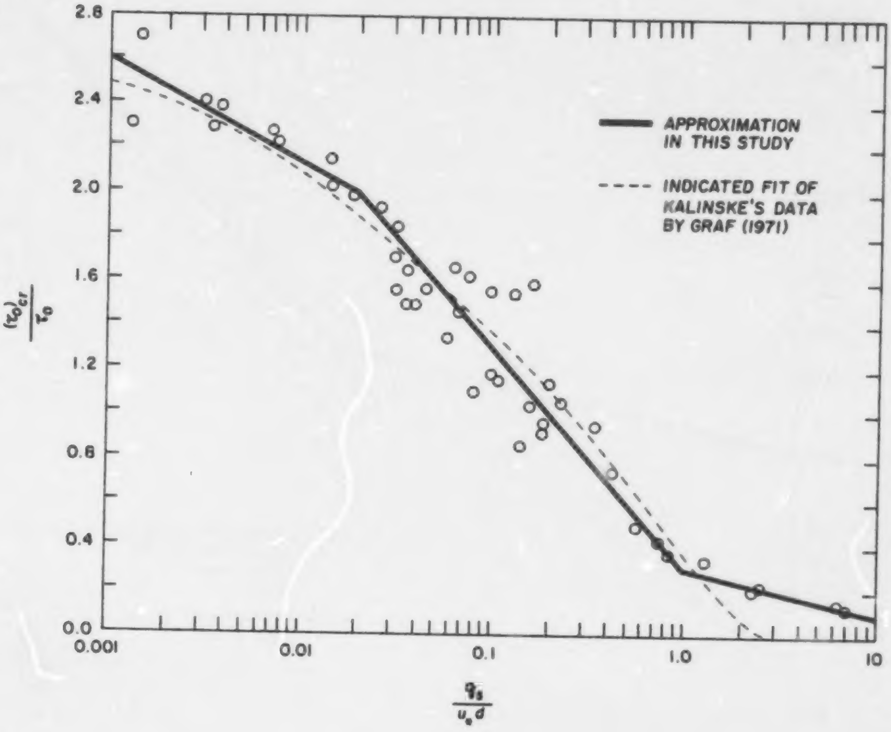


Figure 2
Kalinske's Bedload Function

Again, straight-line approximations have been taken in this study to facilitate the modeling aspects, and indeed to more nearly fit the data in the higher range of q_s/u_*d. The relationships are:

$$\frac{q_s}{u_*d} = 2 \times 10^{-(\tau_o)_{cr}/\tau_o} \qquad 0.3 < (\tau_o)_{cr}/\tau_o \leq 2.0 \qquad [4]$$

$$\frac{q_s}{u_*d} = 434.1 \times 10^{-2.17(\tau_o)_{cr}/\tau_o} \qquad (\tau_o)_c/\tau_o > 2.0 \qquad [5]$$

and

$$\frac{q_s}{u_*d} = 27 \times 10^{-4.77(\tau_o)_{cr}/\tau_o} \qquad (\tau_o)_{cr}/\tau_o \leq 0.3 \qquad [6]$$

The procedure used is as follows:

1) Determine $(\tau_o)_{cr}$ from Figure 1 (or equation 2 or 3).

2) Calculate τ_o from $\gamma\ r_h\ s$.

3) With the ratio of τ's, determine q_s/u_*d from Figure 2 (equation 4, 5, or 6).

4) Compute q_s from $u_*d\ f[(\tau_o)_{cr}/\tau_o]$.

5) Compute the mass in motion per unit width, g_s, as $\gamma_s\ q_s$ where γ_s is the specific weight of the solid particles.

6) Compute the mass in motion, G_s, as $g_s\ b$, where b is the width of the flowing water, estimated as the flow of water, Q, divided by the velocity, V, and the depth, D, all of which are known.

The units of G_s are pounds per second, which multiplied by the length of the time step in seconds, *DELTR*, yields the weight in pounds of bedload of diameter d that *could* move from one junction to the next through the connecting pipe being considered. Because in a sewer the bed is not without limit as in a natural stream, the amount of material transferred from one junction to the next in the model is actually G_s x *DELTR* or the amount available in the bottom deposits that time step, whichever is smaller.

SUSPENDED LOAD TRANSPORT RELATIONSHIPS

As with bedload theory, a plethora of suspended load relationships now exists. Indeed, suspended load transport appears to have received more attention than bedload dynamics. But ASCE (1975) has said explicitly that "it is still not possible to predict the sediment discharge of an alluvial stream with a degree of certainty that is satisfactory for most engineering purposes." However, heeding that caution, we attempted to glean something of value for the pipe problem at hand from the many investigations reported both by ASCE (1975) and by Graf (1971).

Relationships Adopted

The basis for the treatment of suspended load in this study and virtually all others has been the equation attributed to Rouse (Graf, 1971) for the distribution of suspended sediment throughout the flow depth, namely:

$$C_y = C_a \left(\frac{D-y}{y}\,\frac{a}{D-a}\right)^{z} \qquad [7]$$

in which

C_y = the concentration of suspended particles at depth, y in a flow of total depth, D, lb/ft^3;

C_a = a reference concentration near the bed, lb/ft^3;

a = the reference depth where C_a occurs, taken here as $2d_{60}$, ft;

d_{60} = diameter of particles of which 60 percent are finer, ft;

z = v_s/ku_*;

v_s = settling velocity of particles in quiescent water, ft/sec;

k = von Karman "constant"; and

u_* = again, shear velocity $\sqrt{g\, r_h\, s}$, ft/sec.

The intent is to compute a mass rate of movement, g_{ss}, for suspended load, analogous to the bedload mass rate, g_s, from above:

$$g_{ss} = \int_a^D C_y\, u_y\, dy \qquad [8]$$

In this expression, g_{ss} is the mass of suspended load per unit width moving in a unit time, C_y is the concentration computed from equation 7, u_y is the velocity at depth y, and dy is a small interval of depth of which y is the midpoint.

To solve equation 7, values of C_a and z must be found. Much of the work reported by Graf (1971) and ASCE (1975) on suspended load transport has been devoted to evaluation of these two parameters and k, which turns out to be variable and not "constant" at all, although that complication was ignored in this work after some deliberation.

The value of C_a was estimated for each particle size from a relationship given by Graf (1971):

$$C_a = \frac{1}{11.6} \frac{g_s}{u_*' \, a'} \qquad [9]$$

in which

u_*' = shear velocity computed with a hydraulic radius with respect to the grains of solids, $\sqrt{g \, r_h' \, s}$, ft/sec; and

a' = a reference depth, ft, essentially equal to a above.

The value of g_s in this expression is the amount of bedload transport per unit width for each particle size, computed earlier in step 5 of the bedload procedure.

To evaluate u_*', it was necessary to estimate r_h', the portion of the total hydraulic radius attributable to the grains. This was done with an expression from Graf (1971) for velocity distribution which he attributes to Einstein (1950):

$$\frac{V}{u_*} = 5.75 \log \left(12.27 \frac{r_h' \, x}{k_s}\right) \qquad [10]$$

in which V is the average velocity in the pipe, x is a correction factor for wall effects, ranging from about 0.7 to 1.6 and equal to 1.0 for rough wall conditions; and k_s is the Nikuradse sand roughness height, approximately equal to d_{65}. In this study, the value of x was taken as 1.0, and k_s was taken as d_{60} from before. Therefore, the expression used was:

$$\frac{V}{u_*'} = 5.75 \log \left(12.27 \frac{r_h'}{d_{60}}\right) \qquad [11]$$

This equation and $V = \overline{V}$ can be solved iteratively for r_h', since V is defined by equation 11 and is also known from the Hydraulic Model solution, in which the average velocity computed in each pipe is $\overline{V}$. Note also that u_*' was used in equation 11 in place of u_* in equation 10. In summary, r_h' is found in the new model by changing trial values of r_h' until the following function becomes very nearly equal to zero:

$$F = \overline{V} - 5.75 \sqrt{g \, r_h' \, s} \, \log \left(12.27 \frac{r_h'}{d_{60}}\right) \qquad [12]$$

in which $\overline{V}$ is the value of average velocity in the pipe computed in the Hydraulic Model.

With this information, equation 9 can be solved directly for C_a, which is needed in equation 7. The value of z in equation 7 was computed from $v_s/k \, u_*$; where v_s was the settling velocity in still water for each particle size; u_*

was computed with the total hydraulic radius, r_h; and k was assumed to be "constant" indeed and equal to 0.385 (Graf, 1971).

Values of C_y were computed at the midpoints of five equal intervals of depth, from 0 to D. Values of u_y were computed at each interval from velocity distribution equations given by ASCE (1975):

$$u_y = u_* \left[5.5 + 5.75 \log \frac{u_* y}{\nu}\right]; \; R_{10} = \frac{u_* d_{60}}{\nu} \leq 10 \qquad [13]$$

or

$$u_y = u_* \left[8.5 + 5.75 \log \frac{y}{d_{60}}\right]; \; R_{10} = \frac{u_* d_{60}}{\nu} > 10 \qquad [14]$$

in which ν is the kinematic viscosity of water (1.0877×10^{-5} ft²/sec at 20°C).

It was then possible to solve equation 8 numerically as:

$$g_{ss} = \Sigma \left(C_y \, u_y \, \frac{D}{5}\right) \qquad [15]$$

summing over the five depth intervals. As before, the total mass rate of movement was then found by multiplying by width, $G_{ss} = g_{ss} \times b$, and the total mass moved through the pipe in one time step was computed as $G_{ss} \times DELTR$.

Summary of Current Procedure

Lest the reader has become confused, the stepwise procedure used here for computing suspended load is outlined below.

1) Compute $u_* = \sqrt{g \, r_h \, s}$

$a = a' = 2 \, d_{60}$

$R_{10} = u_* \, d_{60}/\nu$ (a Reynolds number)

$k = 0.385$

2) Compute r_h' from

$$F = \overline{V} - 5.75 \sqrt{g \, r_h' \, s} \; \log \left(12.27 \frac{r_h'}{d_{60}}\right) = 0$$

3) Enter particle size loop and compute $z = v_s/k \, u_*$.

4) Compute $C_a = g_s/(11.6 \sqrt{g \, r_h' \, s} \; a')$

5) Enter depth loop and compute

$$u_y = u_* \left[5.5 + 5.75 \log \frac{u_* y}{\nu}\right] \qquad \text{for } R_{10} \leq 10$$

or

$$u_y = u_* \left[8.5 + 5.75 \log \frac{y}{d_{60}}\right] \qquad \text{for } R_{10} > 10$$

6) Compute C_y from equation 7

$$C_y = C_a \left(\frac{D-y}{y}\frac{a}{D-a}\right)^z$$

7) Update g_{ss} and end depth loop or return to step 5.

$$g_{ss} = g_{ss} + C_y u_y \Delta y$$

8) Compute $G_{ss} = g_{ss} b$
and weight of solids moved in the current time step as suspended load in the current size fraction = G_{ss} x *DELTR*.

9) Go forward to perform mass balance for total suspended load moving in the pipe from one junction to the next or return to step 3 for the new particle size and repeat.

WASHLOAD TRANSPORT RELATIONSHIPS

The only work known to the writer on the subject of transient washload estimation is that by Rendon-Herrero (1974) in which the sediment discharge rate is predicted hourly for rural watersheds. It is true that the Universal Soil Loss Equation has been programmed into several models of watershed runoff and used in a transient way, even calibrated in that mode. But that equation was derived to predict average annual loads from a watershed, and its use in dynamic runoff models has not been shown to be valid.

Moreover, the work of Rendon-Herrero (1974), which bears serious consideration for use in a rural watershed runoff model, nonetheless is not applicable in urban areas as it stands; and it only applies to conditions occurring between the watershed boundary and the stream bank. It does not treat transport in the stream itself (much less in pipes).

On the other hand, considerable work has been done on the water-carried transport of bulk materials, such as coal, through pipelines. Graf (1971) devotes an entire section of his book to these investigation. But while the thrust of that work has been toward movement of solids without deposit, precisely our definition of washload in pipes, its purpose has been to define the head loss and related characteristics of the flow that must exist to get the transportation problem solved. It has not been toward predicting a) when material would become washload, or b) how much would move as washload, suspended load, or bedload.

We did adopt one expression from that work, however. Graf (1971) reports a relationship attributable to Durand (1953) for a critical velocity, V_c, separating deposit-free flow from the deposit regime:

$$\frac{V_c}{\sqrt{2gD(S_s-1)}} = F_L \qquad [16]$$

in which

V_c = the critical velocity.

S_s = the specific gravity of the solids, taken as γ_s/γ.

F_L = a parameter ranging from 0.8 to 1.1 for particles larger than 0.5 mm and for concentrations between 2 and 15 percent by weight.

In this study, a value of F_L equal to 0.9 was adopted, which is proper for higher concentrations than normally would be encountered in sewers. A value of 0.8 or even 0.7 would perhaps have been more appropriate. On the other hand, in warning against use of this relationship too rigorously to separate two regimes whose separation is gradual, Graf (1971) suggests using a conservatively high "working" limit velocity, which is exactly what the value of $F_L = 0.9$ will produce. Hence that value is not inappropriate after all.

In any event, this expression can be solved for V_c, the "critical velocity," at which particles will not deposit. That is, their effective settling velocity in the flow can be taken to be zero. At that flow velocity, all particles entering the pipe with a specific gravity of S_s or less will translate down the pipe with the water. At flow velocities less than V_c, a portion of the particles may be able to deposit. When the velocity of the water approaches zero, the pipe will approach an ideal settling tank, and the particles will be removed according to their settling velocity in quiescent water and the detention time in the "tank." For flow velocities between essentially zero and V_c, it can be deduced that the effective settling or deposit velocity of particles will be reduced from its theoretical, quiescent-water value to zero. This information can be used as follows.

The proportion of particles that are removed in a horizontal-flow sedimentation tank is (Fair & Geyer, 1963):

$$PART = \frac{v_s}{Q/A_s} \qquad [17]$$

where v_s is the settling velocity in an ideal, quiescent tank, Q is the flow through the tank, and A_s is the surface area of the tank. In this study the surface area has been computed as b, the width of the flowing water, times the length of the pipe. The flow, Q, is known from the solution in the Hydraulics Model. If the average velocity down the pipe is very small, the effective settling velocity is equal to v_s; and a fraction of solids entering the pipe in the current time period equal to *PART* times the total mass entering is placed in the bed and is then available to be moved as bedload or suspended load, or it may remain at rest. The fraction equal to *1 - PART* is washload, and this mass is moved from one end of the pipe to the other (from the upstream junction to the downstream junction).

If the velocity of flow, V, is equal to or greater than V_c, the effective settling velocity is zero, and all the incoming solids in the particle size under consideration are moved as washload to the next junction.

For velocities, V, between 0.01 ft/sec and V_c, an effective settling velocity is interpolated linearly between 0 and v_s with the expression:

$$w = \frac{V_c - V}{V_c} v_s \qquad [18]$$

Then the value of w is used for v_s in equation 17, and the proportion of solids removed is computed. Therefore, washload is computed as:

$$WL_i = TRANS_i \quad (1 - \frac{w}{Q/A_s}) \qquad [19]$$

where WL_i is the washload in pounds in particle size group i moved through the pipe in the time period, and $TRANS_i$ represents the total solids in pounds in particle size group i that enter the pipe from the upstream (up-gradient) junction in the current time period. Deposition occurs in the model at this point by placing the remainder of $TRANS_i$ in the bed. This is done prior to computing either bedload or suspended load movement, and it is possible that all the particles temporarily placed in the bed by virtue of their being nonwashload are then moved in the same time period as suspended load and/or bedload. These deposited materials cannot become part of the washload in the same time period.

The washload procedure, therefore, follows the following steps:

1) Compute v_s for each particle size.

2) Determine $TRANS_i$ from input runoff pollutographs and dry weather sanitary inputs at the upstream junction.

3) Compute V_c from equation 16.

4) Compute w from equation 18.

5) Compute WL_i from equation 19 and return to step 1 for next particle size or go forward to perform mass balance for total washload moved in the pipe.

SECTION V

THE NEW MODEL

The mathematical model used in this study is an adaptation and expansion of WRE's version of the EPA Storm Water Management Model, known quite widely as SWMM (pronounced "swim"). The original SWMM model (Metcalf & Eddy, *et al.*, 1971) has been applied, updated, and tested extensively (Huber, *et al.*, March 1975; Heaney, *et al.*, May 1975; DiGiano and Mangarella, 1975; Brandstetter, 1976). WRE has developed sewer transport quantity and quality portions of its own for use in the SWMM package (which simulates watershed runoff and receiving water behavior as well) or to be used separately. The WRE version(s) of SWMM have been used in Seattle, San Francisco, and Hamburg, West Germany. This set of models has been reported in several places as well (Kibler, *et al.*, 1973; Kibler, Monser, and Roesner, 1975; Water Resources Engineers, July 1974; Shubinski and Roesner, 1973). The model is available from EPA and WRE's other clients, all of whom are public agencies. Therefore, a full documentation of the model will not be repeated here. Nonetheless, a general, overall description of the WRE Transport Block of the SWMM model is given below.

THE "OLD" MODEL

Overview

The WRE Transport Block is comprised of a quantity section, called the Hydraulics Model, and a quality section or Quality Model. These are separate programs. The Hydraulics Model simulates behavior of flows and heads throughout a pipe network over time and stores the resulting values on tape. These same tapes are then read, along with other card input data, by the Quality Model, which adds and transports or routes masses of various constituents along with the quantities of water and in response to the head differential computed in the Hydraulics Model.

The Hydraulics Model

The Hydraulics Model uses a link-node description of a sewer system. Flows are computed in links, which are the pipes; and heads are computed at nodes, which are the major pipe junctions. Actually nodal volumes are taken as the volumes of all the half-pipe lengths connected to a node.

Links transmit flow from node to node. Properties associated with links are roughness, depth, length, cross-sectional area, hydraulic radius and surface width. The last three properties are functions of the instantaneous depth of flow. The primary dependent variable in the links is the discharge, Q, which is time dependent. Velocity and cross-sectional area of flow, or depth, are also variable with time. Thus the flow condition in a link over a period of many time steps can be both nonuniform and unsteady.

Nodes or junctions are the storage elements of the model and correspond somewhat to manholes or pipe junctions in the physical system. The properties associated with a node are volume, head, and surface area. The primary dependent variable is the head, H, which is assumed to be changing in time but constant throughout a single computational time step. Inflows, such as inlet hydrographs, and outflows such as weir diversions, take place at the nodes of the idealized sewer system. The change in nodal volume during a given time step, Δt, forms the basis of the head and discharge computations.

The equation for describing unsteady flow in a link can be written as:

$$\frac{\partial Q}{\partial t} = - gAS_f + 2V \frac{\partial A}{\partial t} + V^2 \frac{\partial A}{\partial L} - gA \frac{\partial H}{\partial L} \qquad [20]$$

The continuity equation for a node is:

$$\frac{\partial H}{\partial t} = \frac{\Sigma Q}{A_s} \qquad [21]$$

The essence of the Hydraulics Model is to solve these two equations simultaneously, wherein the terms are:

Q = discharge through a conduit or link, ft^3/sec;

V = velocity in the link, ft/sec;

A = cross-sectional area of the link flow, ft^2;

L = length of a link or pipe section, ft;

S_f = the friction slope, ft/ft;

H = hydraulic head, ft;

g = acceleration due to gravity, ft/sec^2;

A_s = water surface area associated with a node, ft^2; and

ΣQ = algebraic sum of the conduit flows entering or leaving a node, ft^3/sec.

The friction slope is defined by the Manning equation, *i.e.*,

$$S_f = \frac{k\, Q|V|}{gA\, r_h^{4/3}}$$

where k is an empirical coefficient equal to gn^2 in the S.I. system, n being the Manning roughness coefficient. In the U.S. Customary system of units:

$$k = \frac{gn^2}{(1.486)^2} = \frac{32.2}{2.208} n^2 = 14.6 n^2$$

which is the system used in the current model. The value of Q in the friction slope equation is the value at the end of the time step, $Q_{t+\Delta t}$.

The finite difference forms of equations 20 and 21 used in the model become equations 22 and 23:

$$Q_{t+\Delta t} = \frac{1}{1 + \frac{k\Delta t|V|}{r_h^{4/3}}} \quad Q_t + 2V\,\Delta A + V^2 \frac{A_2-A_1}{L}\,\Delta t - gA\,\frac{H_2-H_1}{L}\,\Delta t \qquad [22]$$

The values of V, r_h, and A in equation 22 are the weighted average values at the two ends of the conduit at time, t. The subscript 1 refers to the upstream junction, and the subscript 2 refers to the downstream junction. The second equation is:

$$H_{t+\Delta t} = H_t + \frac{\Sigma Q_t \Delta t}{A_{s_t}} \qquad [23]$$

Solutions of equations 22 and 23 are performed by a Modified Euler Method to yield discharges in the links and heads at the junctions at time, $t+\Delta t$.

In addition, the Hydraulics Model detects and accounts for numerous special flow conditions (dry pipes, critical depth occurrences, etc.), surcharge and flooding, and the presence of certain flow control devices such as weirs, orifices, and pump stations, and outfall devices such as tide gates.

The Quality Model

The Quality Model is an advective model which transports individual constituents by advective movement in each conduit. Input masses to be transported are added at junctions. Masses of constituents in runoff from the watershed may be read from a tape generated by a runoff model or computed from hydrograph flow and concentration values read from cards. The dry weather masses of constituents are read from a tape generated in the Hydraulics Model.

The differential equation describing advective transport of quality constituents in a sewer system can be given in either of two forms:

1) Mass rate of change:

$$\frac{\partial M}{\partial t} = QC \qquad [24]$$

where

M = mass, lb;

C = concentration, lb/ft^3;

Q = flow rate, ft^3/sec; and

t = time, sec.

or

2) Concentration rate of change:

$$\frac{\partial C}{\partial t} = -\frac{C}{VOL}\frac{\partial VOL}{\partial t} + V\frac{\partial C}{\partial L} \qquad [25]$$

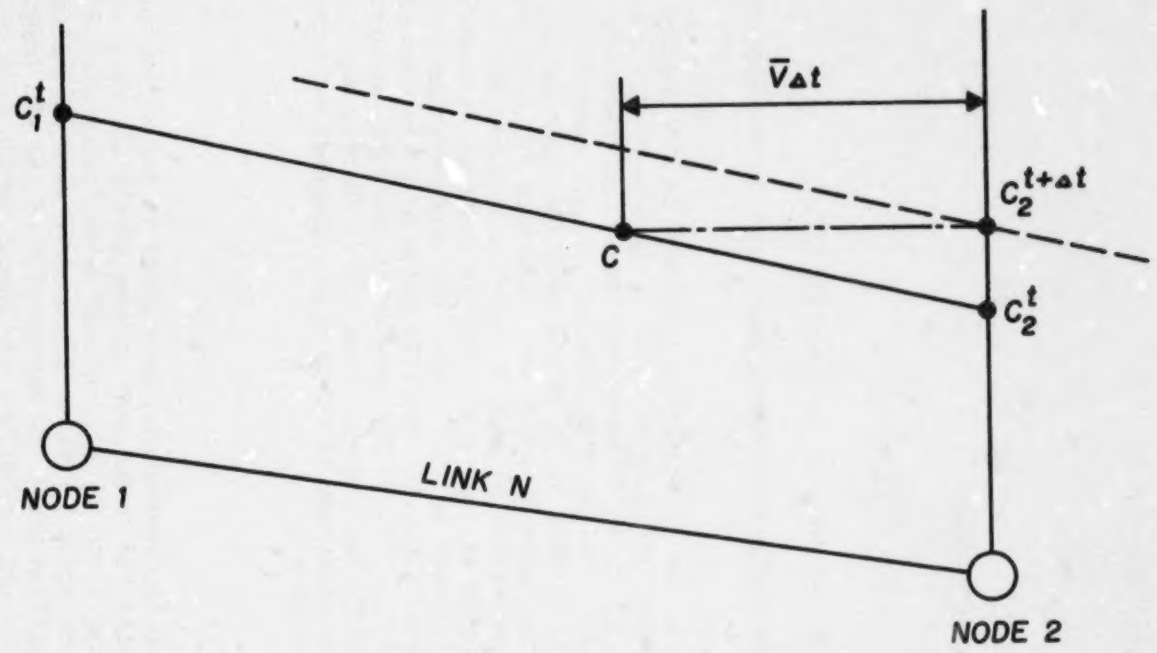
in which VOL is the volume of the node or junction in ft^3.

These equations are interchangeable, and some aspects of both are used in the numerical solution.

The conceptual representation of advective transport in the version of the model adapted here is shown in Figure 3. Link or conduit N with a mean velocity $\bar{V}$ connects nodes 1 and 2, which have constituent concentrations C_1^t and C_2^t, respectively at time t. During time interval Δt, the concentration gradient moves at velocity $\bar{V}$, so that it is in the position shown by the dashed line at time $t+\Delta t$. The value C, which was a distance $\bar{V}\Delta t$ away from node 2, becomes the new concentration at node 2, $C_2^{t+\Delta t}$. This process is repeated for all links and nodes at time $t+\Delta t$, and the sequence is repeated for all time steps in the simulated period. (When the concentration gradient is positive in the direction of flow, the upstream value of C_1^t is used as the new concentration downgradient, $C_2^{t+\Delta t}$.) The version of the model adapted in this study simulated BOD, coliforms, and suspended solids in this way. The major alteration made in this study was to route solids somewhat differently, as described later in this chapter.

NEW ROUTINES

As indicated earlier, no new subroutines were added to the Hydraulics Model, although certain changes had to be made to some of the existing routines. These are discussed later in this chapter under "other changes." The four new separate routines added to the entire package were all added to the Quality Model. Their addition gives the new version of the WRE Transport Block a new kind of capability and generally a new thrust, which is discussed immediately below. Following that, each of the new routines is described in some detail.



C_1^t
$\bar{V}\Delta t$
$C_2^{t+\Delta t}$
C
C_2^t
NODE 1
LINK N
NODE 2

Figure 3
Advective Transport Concept in the Quality Model

The New Overall Capability

Previously, the WRE Transport Block was a simulation tool for analyzing the hydraulic and quality changes in completely mixed flows in sewer systems. A subroutine from the EPA SWMM package had been incorporated to determine when a deposit regime existed and when a scour regime existed. This routine removed solids from the flow during the deposit regime and added solids to the flow during the scour regime; but the resulting masses in suspension were moved conservatively with the water as described earlier, and no net accumulation of material on the pipe bottom was computed.

With the additions made in this project, the model now simulates solids behavior more precisely, it can be used to evaluate the costs of alternative systems of pipes and treatment plants, and it gives detailed reports of the status of the pipe-plant system with respect to solids throughout the simulated period. The model can now be used to analyze such questions as:

1) What are the effects of alternative physical configurations of conduits and treatment on the solids entering and leaving a sewer system, separate or combined, and what are the costs of the alternative systems?

2) In a given situation of new construction, what are the cost and solids removal consequences of using storm and sanitary sewers vs. a combined system?

3) What are the effects of conduit shapes, sizes, and slopes on solids build-up and eventual scour?

4) What are the effects on construction and operation costs resulting from use of various combinations of pipe slope, pipe shape, treatment type, treatment distribution, and treatment capacity? What will be the alternative removal percentages of solids, both through treatment alone and in the overall sense including net deposition during the simulated design period?

Table 1 summarizes the overall capabilities and limitations of the WRE Transport Block.

The model now simulates the transport, deposition, and scour of solids in up to ten different particle size ranges, whose representative diameters are read as input. Particles in each size group are assigned a fixed percentage of the size distribution for sanitary inflows. These separate distributions, percentage wise, are not changed with time. However, composite size distribution is recomputed in each time step, and a weighted specific weight of the solids in each size group is then determined for that time. The specific gravities of each sized particle in dry weather flow must be given, and the specific gravity of each particle size in inflow hydrographs must be given. The Quality Model then routes the composite masses in all ten particle sizes either as washload, bedload, or suspended load, and keeps track of net deposition to or scour from the bottom deposits in each pipe. The other two

TABLE 1. NEW TRANSPORT BLOCK CAPABILITIES AND LIMITATIONS

PIPES OR CHANNELS	
Number	-- Maximum of 100, including up to 60 weirs, 60 orifices, 25 free outfalls, or 20 multi-stage pumps
Flow Characteristics	-- Branched or looped systems, tidal and nontidal backwater, normal flow
Shapes	-- Circular, basket-handle, egg-shaped, horseshoe, rectangular, trapezoidal, cunette sections (3)
Costs	-- Construction, operation, and total annual costs computed in Hydraulics Model
JUNCTIONS	
Number	-- Maximum of 100
Storm Inflows	-- Read from runoff model tape or from cards
Sanitary Inflows	-- Computed in Hydraulics Model as functions of land use
Controls	-- On-line or off-line storage, tide gates, location of surcharged nodes
TREATMENT PLANTS	
Number	-- Maximum of 20
Sizes	-- Any size, specified as design flow in ft^3/sec. (Flow through plant computed each time period; never exceeds design capacity--excess is overflow.)
Types	-- Sedimentation basins, swirl concentrators, sand filtration, primary treatment, secondary treatment
Costs	-- Construction, operation, and total annual costs computed in Quality Model
OPERATIONAL CHARACTERISTICS	
Time Step	-- Hydraulics: usually less than one minute--should be about 15 to 30 seconds
	-- Quality: an even multiple of hydraulic time steps--usually the average L/V
Computer Storage	-- 64,000 words

LIMITATIONS

With the exception of pump facilities, the Hydraulics Model is gravity controlled. Pressurized flow in conduits is not simulated. Also, hydraulic jumps within the system are not simulated.

BOD and coliforms are simulated as conservative, nondecaying constituents. Also, no linkage is made between BOD and solids concentrations; and the treatment simulated removes only solids, not BOD or coliforms.

constituents in the Quality Model, BOD and coliforms, are still routed as described earlier and depicted in Figure 3.

The new version of the Transport Block has been limited in one respect over the older version, in that only 100 pipes and 100 junctions can be included in a single simulation. Previously, up to 230 junctions and 320 pipes were permissible. It was necessary to reduce these dimensions throughout to accommodate new scour and deposition variables that require storage locations, although 100 may not be the true upper limit.

The four new subroutines of the Quality Model are listed in Appendix A of this report. They are described immediately below. Complete lists and card image tapes of both models have been supplied to EPA's Storm and Combined Sewer Section, Edison, New Jersey; and they are available from that source.

A Subroutine for Settling Velocities

In the main program of the Quality Model, in the major loop of time steps and just following the input of new masses at system junctions, the subroutine SETVEL (for "settling velocities") is called to compute the theoretical values of settling velocity in quiescent water.

The methods used in the subroutine and reported here have all been taken from Fair & Geyer (1963).

The subroutine is entered with the diameter of each particle size in feet, d, the weighted specific weight of the solids in that size group, γ_s, the specific weight of water, $\gamma(62.4 \text{ lb/ft}^3)$, the kinematic viscosity, ν, and the acceleration of gravity, g. The purpose of the subroutine is to determine the settling velocity v_s, given that the Reynolds number may be in either the turbulent, transitional, or laminar (Stokes) range--which must be determined.

The subroutine first solves the general expression for settling velocity given by Fair & Geyer (1963):

$$v_s = \sqrt{\frac{4}{3} \frac{gd}{C_D} \frac{(\gamma_s - \gamma)}{\gamma}} \qquad [26]$$

using a value of the drag coefficient, C_D, equal to about 0.4, which applies if the motion is in the turbulent range of Reynolds numbers (*i.e.*, greater than 3000). So the first calculation is for Reynolds number R, to see if it is greater than 3000:

$$R = \frac{v_s d}{\nu} \qquad [27]$$

and if it is, the value of v_s must be correct and control returns to the main program.

If the calculated R is less than 3000, the turbulent condition does not exist, and the trial values of C_D and v_s must therefore be corrected. Consequently, it is assumed next that the settling condition would be in the transitional range of Reynolds numbers (taken as 0.1 to 3000), wherein the drag coefficient can be computed from:

$$C_D = \frac{24}{R} + \frac{3}{\sqrt{R}} + 0.34 \qquad [28]$$

This leads to an iterative solution for v_s, since equation 26 requires C_D and equation 28 requires R, which requires v_s.

So the subroutine attempts first to find a reasonable first trial value of C_D (is it near 1.0, 10, 200?), and having done so it begins an iterative solution via Newton's method to balance the expression:

$$F = C_D - \frac{24\nu}{v_s d} - 3\left(\frac{\nu}{v_s d}\right)^{0.5} - 0.34 = 0 \qquad [29]$$

By this method, the correction to C_D to be made each time period is:

$$C_D = \frac{-F}{dF/dC_D} \qquad [30]$$

The value of dF/dC_D from equation 29, to be used in equation 30, equals unity, so the correction each time is $-F$. In each iteration, a new trial value of v_s is computed with equation 26 and the latest value of C_D. When closure of the function in equation 29 is within .5 percent of the latest value of C_D, the latest trial value of v_s is deemed acceptably accurate, and control returns to the main program.

The remaining possibility is that the settling motion of a particle of this size with this specific weight would be in the laminar range of Reynolds numbers (taken as less than or equal to 0.1). In this case, the subroutine uses equation 26 to compute v_s with the value of C_D taken as $24/R$, where R is the last trial value in the transition range, which by this time must be equal to or less than 0.1. Having completed this task, the subroutine returns control to the main program, which either returns again with a new particle or goes forward to other matters.

The Subroutine for Scour, Deposition, and Solids Transport

The most significant changes made in this project to the Quality Model are embodied in the subroutine called SCRDEP (for "scour and deposition"), which performs the solids routing. This subroutine is called from the main program just following the advective transport of the other constituents, BOD and coliforms.

The subroutine listing in the Appendix should be fairly simple to follow since the names of variables are either the same as the variable names given in Section IV or they are fairly obvious mnemonics, and since the procedure followed is comprised of a small number of separable steps. Those steps follow this sequence:

1) Update the mass of total solids of size, *ISZ*, in potential motion at each junction, *J*, in the current time period. This quantity is *SED(J,ISZ)*. It is equal to the amount present at the junction in the last time period plus the mass added from hydrographs and sanitary inputs in this time period:

$$SED(J,ISZ) = SED(J,ISZ) + MASSIN(J,2) * SPER(J,ISZ) \qquad [31]$$

in which *SED(J,ISZ)* on the right-hand side is the value at the end of the last time period, *MASSIN(J,2)* is the mass of all solids added to the junction in the current time period from external source (the *2* is the constituent counter signifying solids), and *SPER(J,ISZ)* is the fraction of the total mass entering the junction with size, *ISZ*.

2) Enter pipe loop and compute various "constants" for the time period, such as width b, hydraulic slope s, shear stress τ_o, and shear velocity u_*.

3) Determine if treatment is to be applied at the downstream end of the current pipe. If it is, compute the fraction of washload, bedload, and suspended load to be removed there as a function of the treatment process, the capacity of the plant, and the average flow in the pipe during the current time step.

4) Compute the hydraulic radius due to the grains, r_h' (called *RG* in the subroutine), as described in Section IV.

5) Enter particle size loop and compute mass of solids in size *ISZ* that moves into pipe *N* from junction *j* in the current time period. This quantity is:

$$TRANS(ISZ) = SED(J,ISZ) * \frac{|QAVE(N)|DELTR}{VOL(J)} \qquad [32]$$

in which *QAVE(N)* is the flow in pipe *N*, away from junction *J*. Also compute the mass then in pipe *N* available for movement, *AVAIL(ISZ)*, which is equal to *TRANS(ISZ)* plus the amount in the bed, *BED(N,ISZ)*, from the last time step. Next compute the effective settling velocity *W*, from Durand's expression for critical nondeposit velocity and the quiescent water value as described in Section IV.

6) Compute washload mass translated down the pipe *WL*. Place the remainder of *TRANS(ISZ)* in the bed *(BED = BED + OTHER)*, and remove that amount from the particles in motion at the upstream junction *(SED(J,ISZ) = SED(J,ISZ) - OTHER)*. Add

the translated washload, less any removed by treatment, to the downstream junction, k:

$$SED(K,ISZ) = SED(K,ISZ) + WL(ISZ) * (1-R1) \qquad [33]$$

where $R1$ is the fraction of material removed by treatment at the end of pipe N.

7) Compute the bedload transport rate per unit width, GS, as described in Section IV.

8) Compute the suspended load transport rate per unit width, GSS, as described in Section IV.

9) Conduct the sediment balance in mass terms for bedload and suspended load, limiting the amount that can be moved to $AVAIL(ISZ)$, regardless of the sizes of $GS*B*DELTR$ and $GSS*B*DELTR$.

10) End pipe loop and compute the concentrations at the junctions at the end of the time step as:

$$C(J,2) = \sum_{ISZ=1}^{10} SED(J,ISZ) \quad /VOL(J) \qquad [34]$$

11) Return to main program and prepare for printing output.

Figure 4 depicts the basic routing process being used in the model. It might be noted that the bedload is computed prior to suspended load, primarily because GS is needed to compute GSS; but also that both suspended load and bedload are subtracted from the stationary bed. Suspended load is not subtracted merely from bedload, which is in motion. This may appear to be contrary to what we defined suspended load to be in Section IV, namely particles derived solely from bedload.

This ambivalence occurs because if material leaving bedload to become suspended load is immediately replaced by more material from the bed moved into the bedload, then the distinction is academic. No difference in result occurs. If material moved from bedload into suspended load is not replaced in the same time period by material from the bed, then the potential for bedload transport, GS, must be effectively reduced. This seems contrary to the theoretical arguments, since the factors that determine GS, namely u_*, d, τ_o, $(\tau_o)_{cr}$, and γ_s remain constant throughout the time period. Bluntly stated, then, this issue, if it is one, needs more consideration by other scientists; and the dichotomous approach taken here appeals for that.

A Subroutine for Reporting Intermediate Solids Behavior

A relatively short subroutine called SCROUT (for "scour output") has been added to print information on the state of the system with respect to solids during the simulation. Currently this subroutine is called at the first time

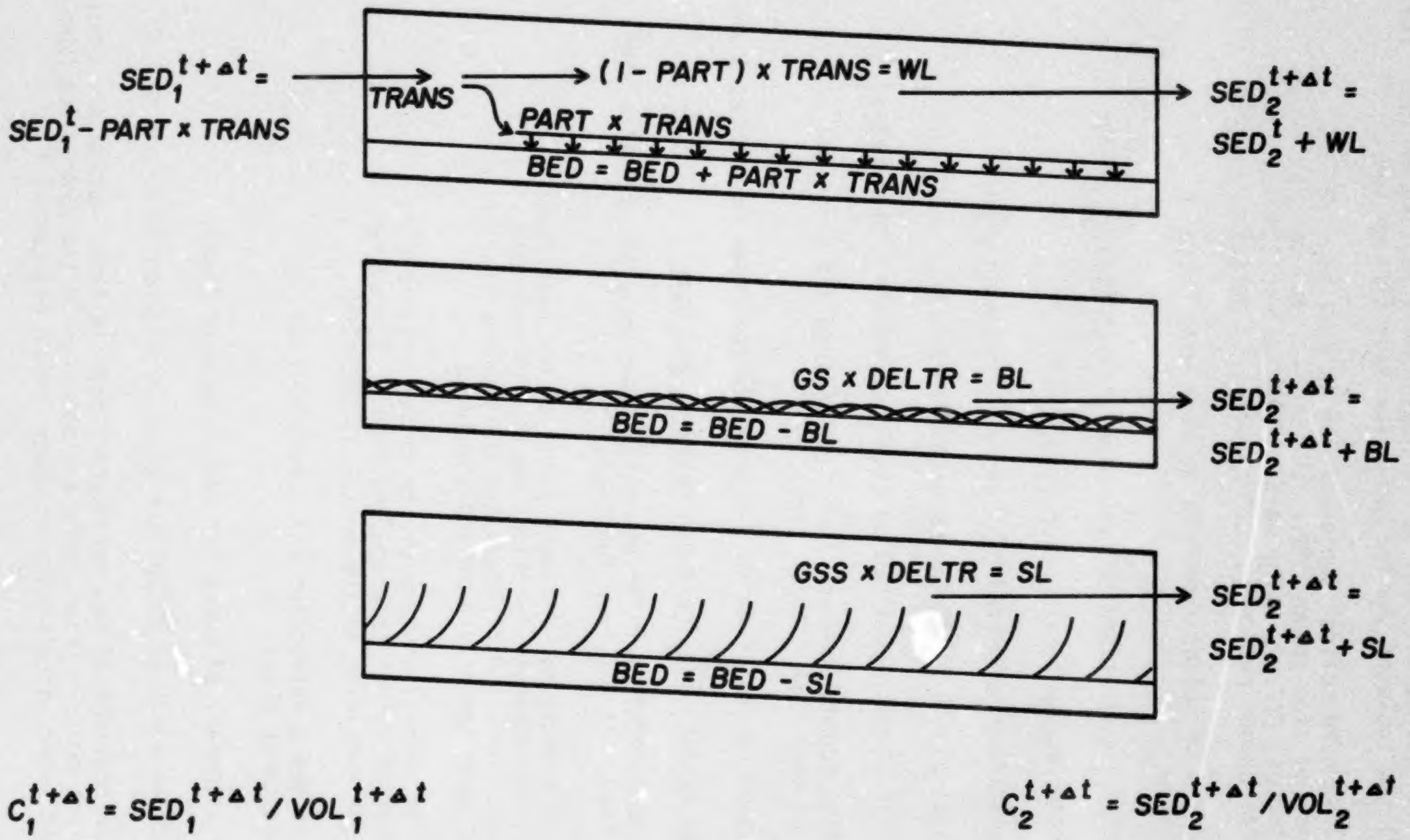

SED1 t+Δt = SED1 t - PART x TRANS
TRANS
(1 - PART) x TRANS = WL
PART x TRANS
BED = BED + PART x TRANS
SED2 t+Δt = SED2 t + WL
GS x DELTR = BL
BED = BED - BL
SED2 t+Δt = SED2 t+Δt + BL
GSS x DELTR = SL
BED = BED - SL
SED2 t+Δt = SED2 t+Δt + SL
C1 t+Δt = SED1 t+Δt / VOL1 t+Δt
C2 t+Δt = SED2 t+Δt / VOL2 t+Δt

Figure 4
Transport Concept for Solids in Subroutine SCRDEP

period, every hour thereafter, and at the last simulated time period. The information printed includes the following for each time period:

1) The "suspended solids" concentration in mg/l at each junction by size group, *SED(J,ISZ)/VOL(J)*, and for all size groups, *C(J,2)*. (The value 0.00006242 in SCROUT converts pounds per cubic foot of water to mg/l.)

2) The pounds of solids remaining on each pipe bottom by size group.

3) The total pounds added to each pipe ("this" time).

4) The total pounds leaving each pipe.

5) The total pounds removed by treatment from each pipe.

6) The total pounds remaining on each pipe bottom.

7) The total pounds deposited on each pipe bottom ("this" time).

8) The total pounds scoured, including bedload and suspended load, in each pipe ("this" time).

Example output reports from this subroutine are shown in Figures 5 and 6.

A Subroutine for Summarizing Treatment Behavior and Cost

The subroutine TREAT (for "treatment") is called after all time steps have been completed. It has four functions:

1) Prints a summary by treatment plant of the capacity, removal percentages (fractions really) of washload, suspended load, and bedload, and the amounts of each removed over the entire period simulated.

2) Prints an overall summary of the system with respect to pounds of solids entering and leaving the system, and those remaining on the bottom of conduits.

3) Computes the construction and operating costs of each treatment plant.

4) Prints a summary of costs for each treatment plant.

Samples of the three kinds of output are given in Figures 7, 8, and 9.

The only computations of new information made in this routine are those for treatment plant costs. (Pipe costs are computed in the Hydraulics Model and are discussed later in this chapter under "other changes".)

```
TIME = 36000.

     SUSPENDED SOLIDS CONC*S.  AT EACH JUNCTION BY SIZE GROUP (ISZ)---MG/L
JUNC       ISZ=1    ISZ=2    ISZ=3    ISZ=4    ISZ=5    ISZ=6    ISZ=7    ISZ=8    ISZ=9    ISZ=10    TOTAL
   1        20.9     20.9     20.9     20.9     20.9     20.9     20.9     20.9     20.9     20.9    209.2
   2        15.6     16.7     15.9     15.1     14.9     14.7     14.7     14.6     14.6     14.6    151.4
   3        14.3     14.4     15.3     14.9     14.9     14.9     14.8     14.8     14.8     14.6    147.7
   4          .0       .0       .2       .2       .2       .2       .2       .2       .2       .2      1.5
   5          .0       .0       .1       .1       .1       .1       .1       .1       .1       .1       .9

     SOLIDS DEPOSITED ON PIPE BOTTOM AT END OF TIME STEP BY SIZE (ISZ)---POUNDS
PIPE       ISZ=1    ISZ=2    ISZ=3    ISZ=4    ISZ=5    ISZ=6    ISZ=7    ISZ=8    ISZ=9    ISZ=10
   1         .00      .00    51.70    65.59    68.36    70.73    72.51    73.52    74.13    74.67
   2         .00      .00      .00    54.81    59.34    63.78    66.91    69.91    72.50    77.21
   3         .00      .00     9.01   113.97   118.53   121.35   124.33   127.08   129.26   132.20
   4         .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
   5         .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
```

Figure 5
Example Output by Particle Size

```
TIME = 36000.

PIPE NO. =        1        LBS ADDED TO PIPE THIS TIME      =        12.4
                           LBS LEAVING PIPE THIS TIME       =          .8
                           LBS REMOVED BY TREATMENT, IF ANY =         5.8
                           LBS REMAINING IN PIPE ON BOTTOM  =       551.2
                           LBS DEPOSITED THIS TIME, IF ANY  =        12.4
                           LBS SCOURED THIS TIME, IF ANY    =         6.5
                           NET LBS SCOURED(-) OR DEPOSITED(+)
                           --NOT INCLUDING WASHLOAD         =         5.8

PIPE NO. =        2        LBS ADDED TO PIPE THIS TIME      =        19.4
                           LBS LEAVING PIPE THIS TIME       =         3.9
                           LBS REMOVED BY TREATMENT, IF ANY =        24.8
                           LBS REMAINING IN PIPE ON BOTTOM  =       464.5
                           LBS DEPOSITED THIS TIME, IF ANY  =        19.4
                           LBS SCOURED THIS TIME, IF ANY    =        28.7
                           NET LBS SCOURED(-) OR DEPOSITED(+)
                           --NOT INCLUDING WASHLOAD         =        -9.2

PIPE NO. =        3        LBS ADDED TO PIPE THIS TIME      =        40.9
                           LBS LEAVING PIPE THIS TIME       =          .5
                           LBS REMOVED BY TREATMENT, IF ANY =        66.2
                           LBS REMAINING IN PIPE ON BOTTOM  =       875.7
                           LBS DEPOSITED THIS TIME, IF ANY  =        39.6
                           LBS SCOURED THIS TIME, IF ANY    =        65.4
                           NET LBS SCOURED(-) OR DEPOSITED(+)
                           --NOT INCLUDING WASHLOAD         =       -25.8

PIPE NO. =        4        LBS ADDED TO PIPE THIS TIME      =          .4
                           LBS LEAVING PIPE THIS TIME       =          .4
                           LBS REMOVED BY TREATMENT, IF ANY =          .0
                           LBS REMAINING IN PIPE ON BOTTOM  =          .0
                           LBS DEPOSITED THIS TIME, IF ANY  =          .0
                           LBS SCOURED THIS TIME, IF ANY    =          .0
                           NET LBS SCOURED(-) OR DEPOSITED(+)
                           --NOT INCLUDING WASHLOAD         =          .0
```

Figure 6
Example Output by Pipe

```
PLANT NUMBER 1 AT END OF PIPE 1
       DESIGN FLOW = 100.0 FT3/SEC
       WASHLOAD REMOVAL PERCENTAGE                  =           1.0
       SUSPENDED LOAD REMOVAL PERCENTAGE            =            .9
       BEDLOAD REMOVAL PERCENTAGE                   =            .9

       WASHLOAD REMOVED DURING EVENT                =          43.LBS
       SUSPENDED LOAD REMOVED DURING EVENT          =        1000.LBS
       BEDLOAD REMOVED DURING EVENT                 =        3176.LBS

       TOTAL LOAD REMOVED DURING EVENT              =        4220.LBS


PLANT NUMBER 2 AT END OF PIPE 2
       DESIGN FLOW = 150.0 FT3/SEC

       WASHLOAD REMOVAL PERCENTAGE                  =           1.0
       SUSPENDED LOAD REMOVAL PERCENTAGE            =            .9
       BEDLOAD REMOVAL PERCENTAGE                   =            .9

       WASHLOAD REMOVED DURING EVENT                =          53.LBS
       SUSPENDED LOAD REMOVED DURING EVENT          =        1673.LBS
       BEDLOAD REMOVED DURING EVENT                 =        5485.LBS

       TOTAL LOAD REMOVED DURING EVENT              =        7210.LBS


PLANT NUMBER 3 AT END OF PIPE 3
       DESIGN FLOW = 32.0 FT3/SEC
       WASHLOAD REMOVAL PERCENTAGE                  =           1.0
       SUSPENDED LOAD REMOVAL PERCENTAGE            =           1.0
       BEDLOAD REMOVAL PERCENTAGE                   =           1.0

       WASHLOAD REMOVED DURING EVENT                =         472.LBS
       SUSPENDED LOAD REMOVED DURING EVENT          =        2662.LBS
       BEDLOAD REMOVED DURING EVENT                 =        7162.LBS

       TOTAL LOAD REMOVED DURING EVENT              =       10296.LBS
```

Figure 7
Example Output by Treatment Plant

```
                    * * * OVERALL SYSTEM SUMMARY * * *

POUNDS OF SOLIDS ON BOTTOM AT BEGINNING OF EVENT      =             0.

POUNDS OF SOLIDS ON BOTTOM AT END OF EVENT            =          2147.

POUNDS ENTERING THE SYSTEM DURING THE EVENT           =         27605.

POUNDS DISCHARGED FROM SYSTEM DURING EVENT            =          3731.
                                 (CHECK VALUE )       =          3732.)

POUNDS REMOVED BY TREATMENT DURING THE EVENT          =         21725.

NET POUNDS SCOURED(-) OR DEPOSITED(+) IN EVENT        =          2148.

NET POUNDS REMOVED (DEPOSITION + TREATMENT)           =         23873.

                    OVERALL PERCENTAGE REMOVAL        =           86.5
```

Figure 8
Example Output for Entire System

```
                              SUMMARY OF TREATMENT COSTS
                     (PIPE COSTS FROM TRANSPORT MODEL MUST BE ADDED)

TREATMENT        TREATMENT       CONSTRUCTION       ANN. CONS.      ANN OPER.       TOTAL ANN.
PLANT NO.          TYPE          COST, DOLLARS      COST, $/YR      COST, $/YR      COST, $/YR

    1                3              234551.           16419.          3262.           19680.
    2                3              292554.           20479.          3691.           24170.
    3                4              726223.           50836.         12327.           63162.

                          TOTALS   1253328.           87733.         19280.          107013.
```

Figure 9
Example Output of Treatment Costs

The cost equations used in subroutine TREAT have been derived from two sources (Culp, Wesner, and Culp, 1976; Montgomery-WRE, 1974), and these sources gave general relationships for cost versus plant capacity. Consequently, the cost relationships reported here are worthwhile for preliminary planning purposes only. However, the equations used have been generalized such that only the coefficients, which are read as data, might have to be altered from application to application.

In any event, the cost equations used are as follows. The capital cost equation is:

$$DOL_m = ACC_k \times QDES_m^{BCC_k} \qquad [35]$$

where

DOL_m = the construction cost of plant number, m;

ACC_k = a coefficient for plants of type, k;

$QDES_m$ = design capacity of plant m, ft^3/sec; and

BCC_k = an exponent for plants of type, k.

The annual operation and maintenance cost is computed with a similar equation that uses the average amount of water treated rather than the design capacity.

$$DOLY_m = OM1_k \times QTRT_m^{OB1_k} + OM2_k \times QTRT_m^{OB2_k} + OM3_k \times QTRT^{OB3_k} + OM4_k \qquad [36]$$

in which

$DOLY_m$ = annual operating cost for plant m, \$/yr,

$OM1_k$, $OM2_k$, etc. = coefficients for plants of type, k;

$OB1_k$, $OB2_k$, etc. = exponents for plants of type, k; and

$QTRT_m$ = average annual flow through the plant, ft^3/sec., taken here as the minimum of $QDES_m$ and the largest $QAVE(N)$ attained in the pipe upstream of the plant at any time during the simulated period.

Values of the coefficients and exponents for the various kinds of treatment considered are shown in Table 2.

OTHER CHANGES TO THE TRANSPORT BLOCK

The two major changes made in this project other than the new subroutines in the Quality Model were the addition of conduit cost calculations in the

TABLE 2. TREATMENT COST COEFFICIENTS AND EXPONENTS

Treatment Type	k	ACC	BCC	OM1	OB1	OM2	OB2	OM3	OB3	OM4
None	1			(NO	DATA	REQUIRED)				
Sedimentation	2	111,749	0.8725	1,202	0.222	1.65	0.933	--[1]	0.778	0
Swirl Conc.	3	19,065	0.545	--[2]	0.273	0	0	0	0	1,760
Filtration	4	76,919	0.6478	656.5	0.2275	8.43	0.903	--[3]	0.100	0
Primary	5	345,000	0.85	29,600	0.690	0	0	0	0	0
Secondary[4]	6	322,570	0.8184	36,570	0.5258	0	0	0	0	0

	Values for Various Overflows per Years 5	10	20	30
[1] $OM3_2$	712	1,160	2,100	3,220
[2] $OM1_3$	349	788	1,369	1,944
[3] $OM3_4$	2,940	5,310	9,150	15,430

[4] Construction and operations costs for secondary treatment must be added to costs computed for primary treatment.

Hydraulics Model and the addition of capability throughout both models to have cunette sections in conduits.

Conduit Cost Equations

Pipeline costs are computed and printed in the INDATA (for "input data") subroutine of the Hydraulics Model. The cost equations used were derived from those reported in the Seattle RIBCO study (Water Resources Engineers, December 1974).

The cost equations taken from the Corps of Engineers study in Seattle (Water Resources Engineers, December 1974) were for the cost per foot of circular "Class V concrete" pipe, and an ENR index of 1450. In this study the equations have been altered to account for different shapes and for an ENR index of 2000 (which may now be generally too low).

The old equations for cost in dollars per foot of circular pipe were:

$$\$/ft = 0.586\ DIAM^{1.15}\ Z^{0.118} \qquad Z \le 12' \qquad [37]$$

and

$$\$/ft = 0.246\ DIAM^{1.15}\ Z^{0.45} \qquad Z > 12' \qquad [38]$$

in which

$DIAM$ = diameter of circular pipe in inches, and

Z = feet of required excavation.

The equations as used in the model were updated for an ENR index of 2000, then increased by 25 percent to include cost items explicitly not included in the old equations, and given a coefficient to account for higher costs associated with constructing conduits of irregular shapes. The equations for cost in dollars are now:

$$DOL = ACOF_k \times DIAM^{1.15}\ Z^{0.118}\ LEN_n \qquad Z \le 12'$$

and

$$DOL = 0.42\ ACOF_k \times DIAM^{1.15}\ Z^{0.45}\ LEN_n \qquad Z > 12'$$

in which

$ACOF_k$ = a coefficient to increase cost for noncircular pipes,

LEN_n = the length of conduit n in feet, and

$DIAM$ = the diameter in inches of equivalent circular pipe (computed from the cross-sectional area of the full conduit).

The values of the coefficient $ACOF_k$ used in the model are as follows:

1) Circular pipe--1.0

2) Rectangular pipe--2.0

3) Horseshoe pipe--1.5

4) Eggshaped pipe--1.5

5) Baskethandle pipe--1.5

6) Trapezoids--2.0

7) Circle with semi-circular cunette--1.25

8) Circle with rectangular cunette--1.25

9) Rectangle with rectangular cunette--2.0

These values were our own estimates, although construction cost information for only one noncircular shape (type 7) were available for corroboration.

The operation and maintenance costs were estimated as 0.5 percent per year of the construction cost (Montgomery-WRE, 1974). The cunette section operating costs were reduced by 50 percent from that estimated simply because they are supposed to be more self-cleansing by definition, and hence require less maintenance (cleaning).

The Hydraulics of Cunette Sections

Throughout both the Hydraulics Model and the Quality Model some additional computations were programmed to account for the three new cunette-shaped cross-sections.

In the former programs the values of hydraulic radius, surface width, and cross-sectional area were computed from ratios of these properties at 25 depth intervals to those same properties at full flow. The values of the ratios were built into the programs. The precise values were found by interpolation between the two values in the table bracketing the computed depth.

Now, in the updated programs, these tabulated ratios are still used when the flow is in the cunette of a cunette-shaped conduit. The computations at these depths are equally as precise as before.

When the flow is above the cunette, that is in the large barrel, the cross-section is changed to a regular circle or rectangle having a full cross-sectional area equal to the full cunette-large-barrel section, and the built-in tables are used as before to find the hydraulic radius, surface width, and cross-sectional area at the less than full depths. This approximation can result in inaccurate computations of velocity when the flow is just barely above the top of the cunette in the large barrel. When flow is considerably above the cunette, this approximation works fairly accurately. In the example cases reported subsequently, no problems arose as a result of this simplification.

To avoid this inaccuracy, the programs could be altered to compute the geometric properties of the cunette sections exactly. The alternatives are

a) to recompute the properties from geometric relationships in each time period, which we felt would require too much computer time; b) compute and save a tabulated set of depth-to-property ratios in each simulation for each cunette section in the system being studied, which we predicted could require excessive storage; or c) build a single table of ratios of depth to other geometric properties, assuming all cunettes and large barrels would have relative dimensions that were constant, which we simply did not want to do.

The three shapes and the variables that define their dimensions are shown in Figure 10. The value of *STHETA* is dimensionless and is the ratio of the cunette-to-large-barrel depth. This value should probably be equal to or less than 0.25. The value of *SPHI* is given in feet, and must simply be smaller than *WIDE*. These values are read in the Hydraulics Model on a card that defines the length, Manning's n, and other properties of each conduit.

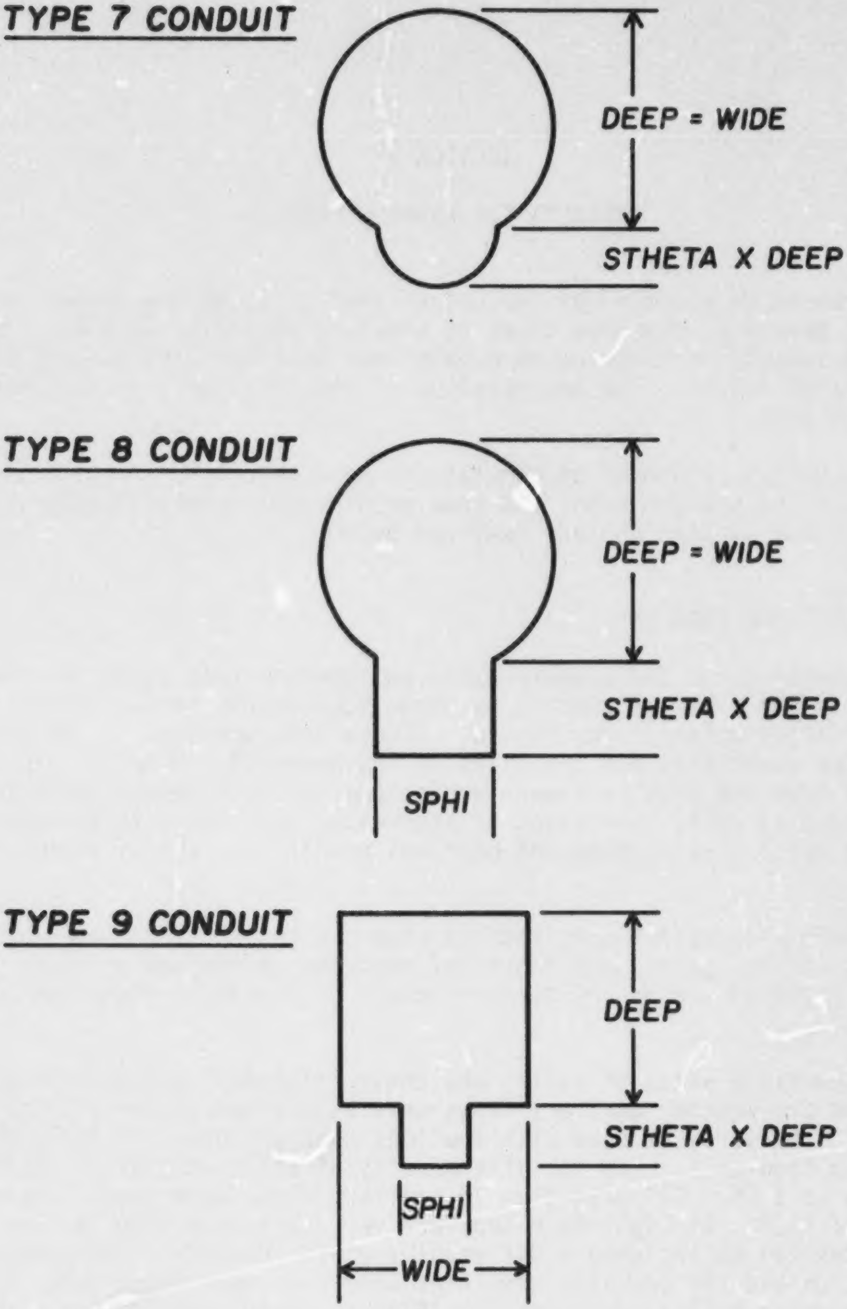

TYPE 7 CONDUIT
DEEP = WIDE
STHETA X DEEP
TYPE 8 CONDUIT
DEEP = WIDE
STHETA X DEEP
SPHI
TYPE 9 CONDUIT
DEEP
STHETA X DEEP
SPHI
WIDE

Figure 10
Three Conduit Shapes Added to WRE-SWMM Model

SECTION VI

RESULTS FOR EXAMPLE PROBLEM

It should be stressed at the outset that although the former Transport Block has been used numerous times to simulate real pipe networks, the new scour and deposition features have only been executed with the hypothetical examples that follow. The new versions of the programs have not been verified with field data.

Nonetheless, a number of simulations have been made to check the executability of the new programs, and some predictable or explainable results have ensued. These simulations are reported below.

THE HYPOTHETICAL PROBLEM

The setting for the example cases analyzed in this study is shown in Figure 11. There are assumed to be three subareas in series having the characteristics noted in the Figure. (These data are used in computing the dry-weather quantities and qualities in the Hydraulics Model.) The question is: What type and size of sewers would make the most sense, considering factors such as cost, quantities of stormwater and sewage to be handled, treatment options available, and physical conditions such as slope and land area?

Example simulations were made to show the results achieved with different slopes, sizes, and shapes of conduits, storm and sanitary sewers, a single combined sewer, and various levels of treatment installed at various locations.

The particle sizes of solids and their fractional portions in storm runoff and dry-weather sanitary flows were held constant for all the cases examined. The particle size distributions used are given in Table 3 and are plotted in Figure 12. The specific gravity of all particles in storm runoff was given as 2.65. All particles in sanitary flows were given a specific gravity of 1.20. It might be noted, however, that each size in each of the two sources can be assigned a different specific gravity. The particle size distribution and the specific gravities used here were those used by Sullivan and his coworkers (Sullivan, *et al.*, 1974) in their study of swirl concentrators.

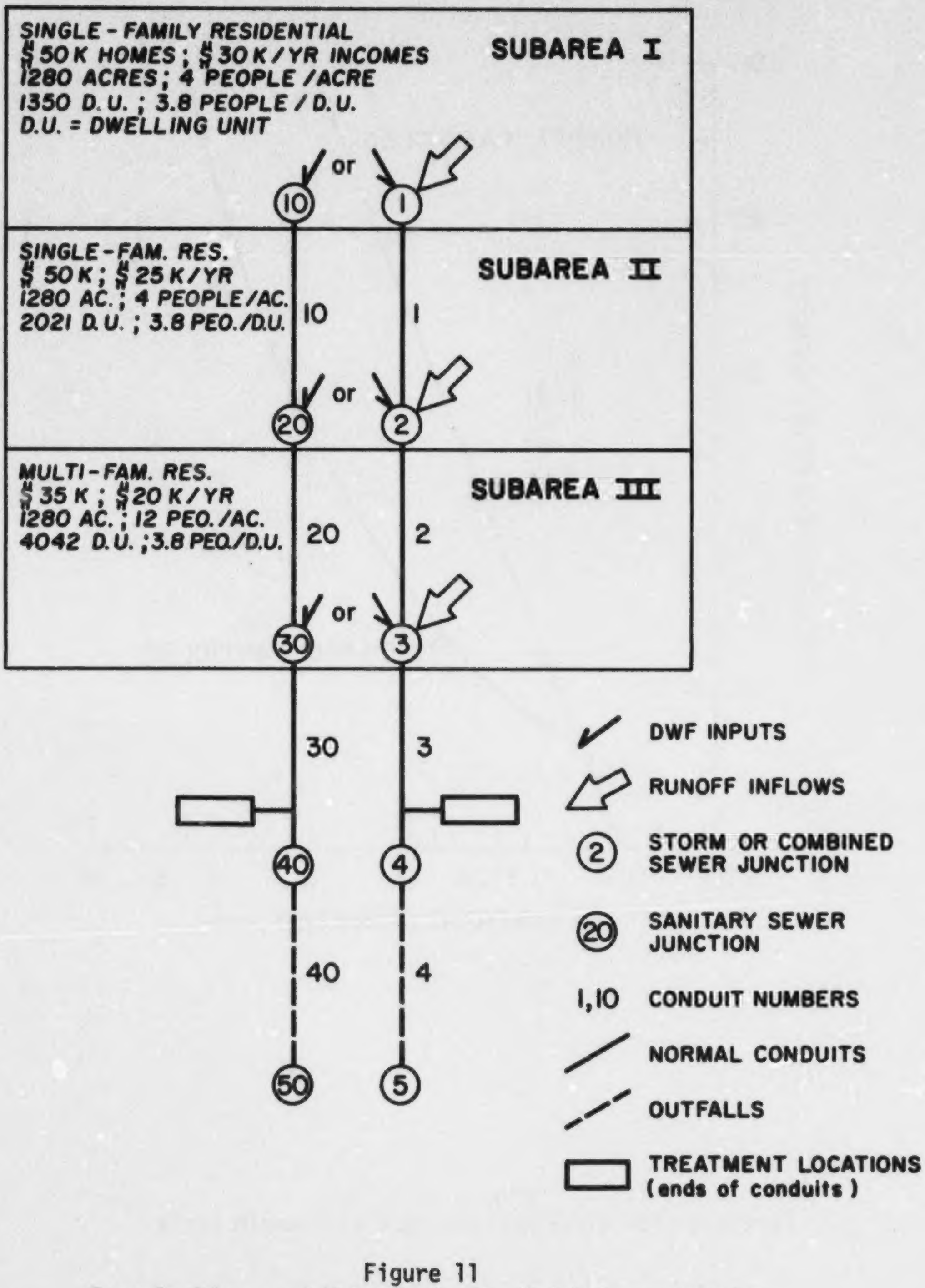

SINGLE-FAMILY RESIDENTIAL
$50 K HOMES; $30 K/YR INCOMES
1280 ACRES; 4 PEOPLE/ACRE
1350 D.U.; 3.8 PEOPLE/D.U.
D.U. = DWELLING UNIT
SUBAREA I
or
10
1
SINGLE-FAM. RES.
$50 K; $25 K/YR
1280 AC.; 4 PEOPLE/AC.
2021 D.U.; 3.8 PEO./D.U.
SUBAREA II
10
1
or
20
2
MULTI-FAM. RES.
$35 K; $20 K/YR
1280 AC.; 12 PEO./AC.
4042 D.U.; 3.8 PEO./D.U.
SUBAREA III
20
2
or
30
3
30
3
40
4
40
4
50
5
DWF INPUTS
RUNOFF INFLOWS
2 STORM OR COMBINED SEWER JUNCTION
20 SANITARY SEWER JUNCTION
1,10 CONDUIT NUMBERS
NORMAL CONDUITS
OUTFALLS
TREATMENT LOCATIONS (ends of conduits)

Figure 11
Example Pipes and Watersheds for Model Demonstration

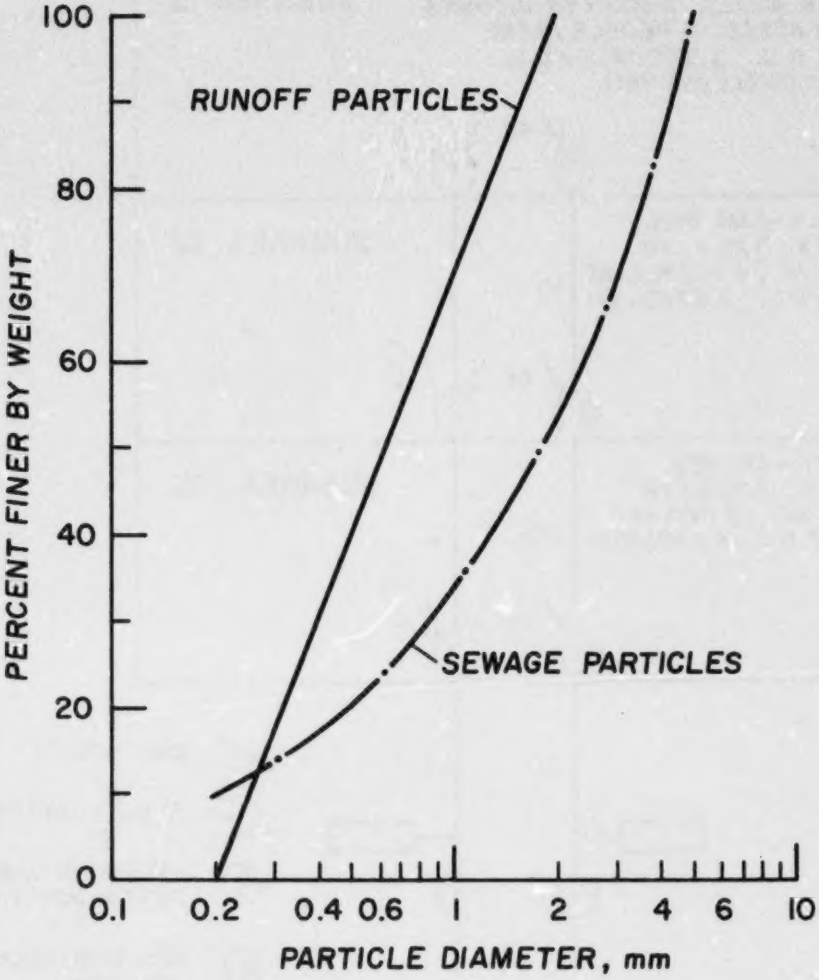


Figure 12
Particle Size Distributions Used in Example Cases

TABLE 3. PARTICLE SIZE DISTRIBUTIONS

Variable	Particle Size Groups (ISZ)									
	1	2	3	4	5	6	7	8	9	10
Representative Diameter, mm-DSOL(ISZ)	0.12	0.32	0.63	1.05	1.50	2.1	2.5	3.2	3.7	4.4
Stormwater										
Fraction of total input solids in each size group	--	0.22	0.30	0.21	.185	.085	--	--	--	--
Sanitary Inflows										
Fraction of total input solids in each size group	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1

EXAMPLE CASES

The 16 basic example cases are described briefly in Table 4. The pipe and junction numbers are those given in Figure 11 previously.

The sizes of the conduits used in the various example simulations are given in Table 5. The elevations of the junction inverts are given in Table 6. The Hydraulics Model used these data to compute pipe slopes. The intent was to have the outfalls drop 10 feet in 5,000 feet, and the remainder of the pipes were to drop between roughly 4 and 6 feet in 5,000 feet. Hence, the slopes of the pipes, other than outfalls, ranged from 0.0008 to 0.00125 foot per foot (0.08 to 0.125 percent). Very low slopes were desirable in these examples to permit deposition to occur at low flows, depths, and velocities. All pipes were 5,000 feet long.

The treatment sizes and types were varied as shown in Table 7. Since treatment sizing was not a critical part of the exercise, the treatment devices were assigned capacities computed from crude rules of thumb. The dry-weather hydraulic capacity was computed as 4 times the average daily flow (4 cfs x 4 = 16 cfs), or about twice the normal peak hour flow. The value of 25 cfs for the combined system was chosen as a conveniently larger number to allow for some downstream treatment capacity for stormwater. The swirl concentrator capacities were selected between 50 and 150 cfs to show the effects of bypassing much of the flow in some cases and treating most of the flow in others; since the assumed input hydrographs rose from zero to a peak of 100 cfs, and there were several of these hydrographs added at upstream storm and combined sewer junctions (numbers 1, 2, and 3 in Figure 11).

The assumed input hydrographs and solids concentrations for all cases are shown in Figure 13. The flow peaked at the first hour after runoff began. For all but one case, the runoff was started at all three upstream junctions at 1:00 a.m., one hour after the beginning of the simulation. The dry weather inputs computed in the Hydraulics Model are given in Figure 14.

From the data presented, one can discern that the points of the examples were these:

1) Cases I-III and VII-IX were to evaluate the effect of slope changes in combined and separate systems, respectively.

2) Cases IV and X were to evaluate the effects of upstream treatment of stormwater.

3) Cases V and XI were to evaluate increases in degree of treatment for dry-weather or sanitary flows.

4) Cases VI and XII were to evaluate the effect of enlarging and upgrading treatment generally.

5) The cunette cases were to evaluate the effects of the shape changes to conduits.

TABLE 4. DESCRIPTION OF EXAMPLE CASES

Example Case	Description
I	Circ. pipes--combined sys.--25 cfs pri. treat. @ end of pipe 3--slope = S
II	Same as I--except slope = 1.5 S
III	Same as I--except slope = 1.25 S
IV	Same as II--with swirls at pipes 1 and 2
V	Same as II--except treatment = secondary
VI	Slope = 1.5 S--circ. pipes, 1 foot smaller; larger swirls at 1 & 2; filtration plant at 3--cost added to secondary of Case V.
Cunettes-combined	Circ. pipes w/semi-circ. cunettes. Lg. barrels same as pipes in I-V--STHETA = 0.1667--slope = 1.25 S
Cunettes-combined-rect.	Rect. pipes w/rect. cunettes--STHETA = 0.4--SPHI = 2 ft. and 2.8 ft.--slope = 1.25 S
VII	Circ. pipes--sep. system--16 cfs pri. treat. @ end of pipe 30--slope = S
VIII	Same as VII--except slope = 1.5 S
IX	Same as VII--except slope = 1.25 S
X	Same as VIII--except swirls at pipes 1 and 2
XI	Same as VIII--except treatment = secondary
XII	Slope = 1.5 S, all pipes smaller, larger swirls at 1 & 2--16 cfs. filtration plant at pipe 30
Cunettes-sep.	Same as Cunettes-combined, except san. sys. added
Cunettes-sep.-enl.	Large barrels enlarged to avoid surcharge of some nodes

TABLE 5. CONDUIT SIZES IN EXAMPLE CASES

Example Case	Sizes of Following Conduits, feet							
	1	2	3	4	10	20	30	40
I	6	7	8	8				
II	6	7	8	8				
III	6	7	8	8				
IV	6	7	8	8				
V	6	7	8	8				
VI	5	6	7	7				
Cunettes-comb.[1]	6	7	8	8				
Cunettes-rect.[2]	5	7	7	7				
VII	5	6	7	7	1.667	2.5	3	3
VIII	5	6	7	7	1.667	2.5	3	3
IX	5	6	7	7	1.667	2.5	3	3
X	5	6	7	7	1.667	2.5	3	3
XI	5	6	7	7	1.667	2.5	3	3
XII	4.5	5.5	6.5	6.5	1.5	2	2.5	2.5
Cunettes-sep.[3]	5	6	7	7	1.667	2.5	3	3
Cunettes-enl.[4]	6	7	8	7	1.667	2.5	3	3

---

[1] Large barrel sizes given for all cunette sections--STHETA = 0.1667

[2] STHETA = 0.4; SPHI = 2, 2.8, 2.8, 2.8, respectively

[3] STHETA = 0.1667

[4] STHETA = 0.25 (all pipes)

TABLE 6. JUNCTION INVERT ELEVATIONS FOR EXAMPLE CASES

Example Case	Invert Elevations at Following Junctions									
	1	2	3	4	5	10	20	30	40	50
I	23.5	18.5	14	10	0					
II	30.25	22.75	16	10	0					
III	26.875	20.625	15	10	0					
IV	30.25	22.75	16	10	0					
V	30.25	22.75	16	10	0					
VI	31.25	23.75	17	11	0					
Cunettes-comb.	25.54	19.29	13.67	8.67	-1.33					
Cunettes-rect.	25.54	19.29	13.67	8.67	-1.33					
VII	24.5	19.5	15	11	1	28.5	23.5	19	15	5
VIII	31.25	23.75	17	11	1	35.25	27.75	21	15	5
IX	27.875	21.625	16	11	1	31.875	25.625	20	15	5
X	31.25	23.75	17	11	1	35.25	27.75	21	15	5
XI	31.25	23.75	17	11	1	35.25	27.75	21	15	5
XII	31.75	24.25	17.5	12.5	1	35.42	28.25	21.5	15.5	5
Cunettes-sep.	26.708	20.458	14.833	9.833	-0.167	31.375	25.125	19.50	14.50	4.50
Cunettes-enl.	26.708	20.458	14.833	9.833	-0.167	31.375	25.125	19.50	14.50	4.50

TABLE 7. TREATMENT SIZES AND TYPES IN EXAMPLE CASES

Example Case	Treatment Design Capacities, cfs, (and Types*) at Ends of Following Pipes							
	1	2	3	4	10	20	30	40
I			25(5)					
II			25(5)					
III			25(5)					
IV	50(3)	100(3)	25(5)					
V			25(6)					
VI	100(3)	150(3)	50(4)					
Cunettes-comb.			25(5)					
Cunettes-rect.			16(5)					
VII							16(5)	
VIII							16(5)	
IX							16(5)	
X	50(3)	100(3)					16(5)	
XI							16(6)	
XII	100(3)	150(3)					16(4)	
Cunettes-sep.							16(5)	
Cunettes-enl.							16(5)	

*3 = swirl concentrator; 4 = filtration, added to secondary;
5 = primary; 6 = secondary

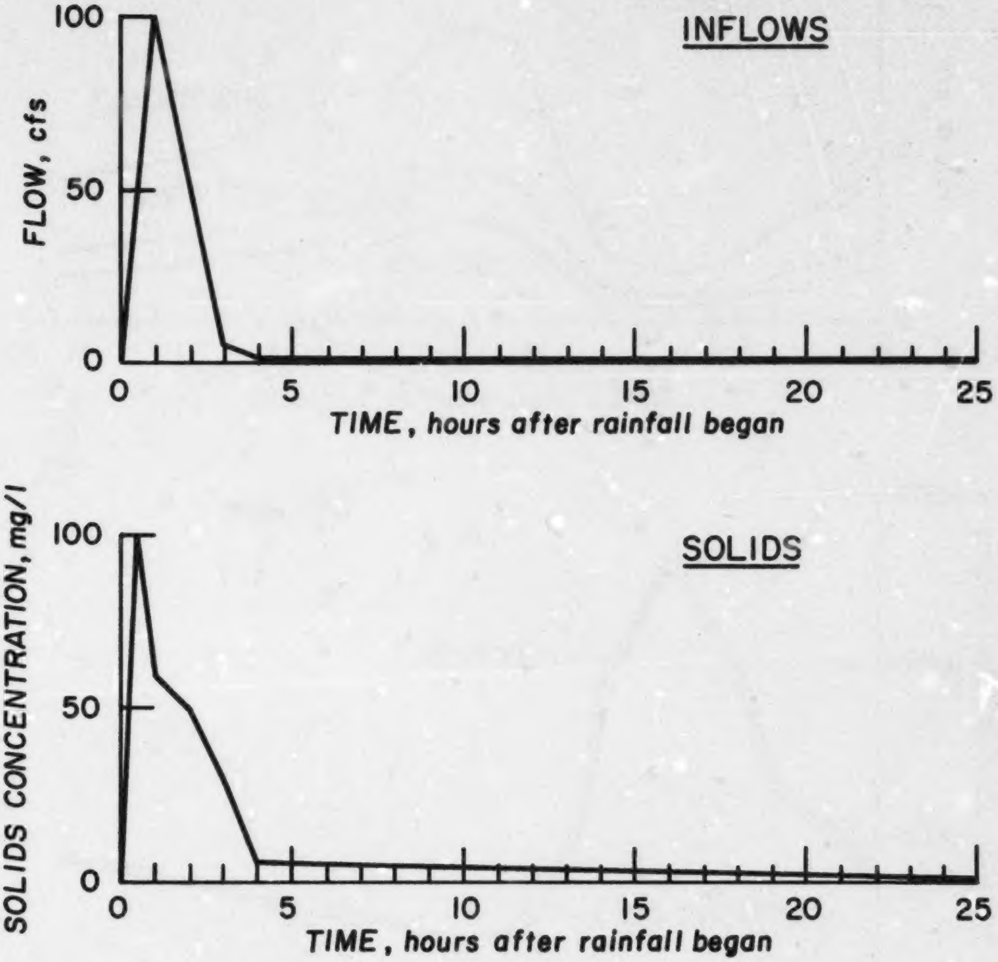


Figure 13
Storm Inputs at Junctions 1, 2, and 3 for All Example Cases

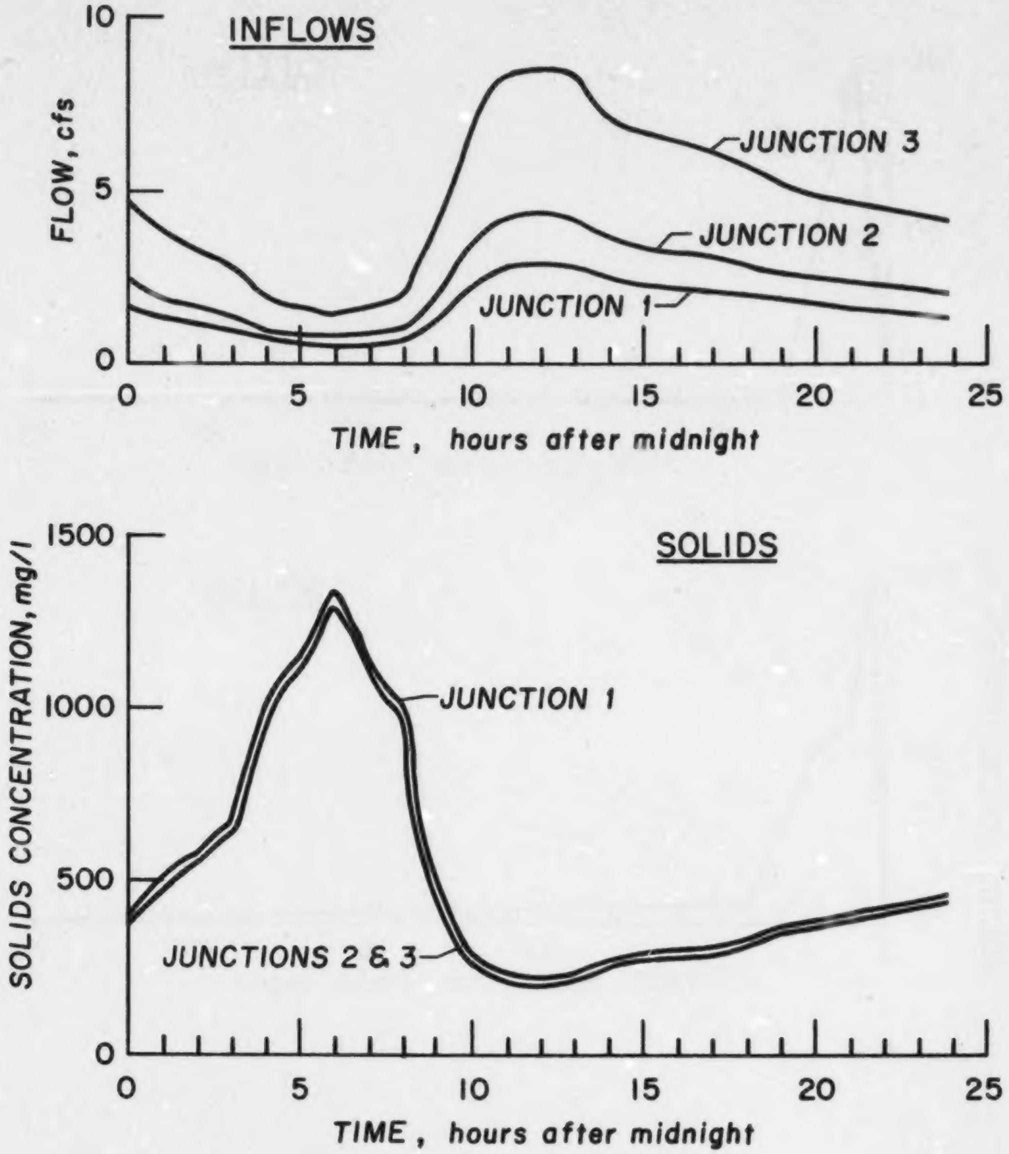


Figure 14
Dry Weather Inputs for All Example Cases

DEMONSTRATION OF FIRST FLUSH

In the cases just itemized the input hydrographs were timed to begin one hour after the beginning of the simulation. The result was that little deposition had occurred prior to the storm, which washed the small deposits away rather quickly. Moreover, as shown in Figure 14, the solids concentrations in the dry-weather flow were highest just after this period anyway; so the differences between what was caused by the storm and what was caused by the dry weather conditions were not easily discernible.

Consequently, a separate simulation was made for the Case III circular combined sewer network with moderate slopes, but with the hydrographs delayed until the 10th hour of the simulated day. This allowed deposition to occur during the low-flow, high-concentration period, and made the flushing by the high flows more dramatically discernible. Since the simulation of first flush was a major point of the model development exercise, those results are presented here in some detail.

First, the flows and velocities in the 4 pipes are shown in Figure 15. There was roughly a 10-20 fold increase in flow and a 3-4 fold increase in velocity from hour 10 to hour 11. The peak flow and velocity occurred in pipe 1 at 11:00 o'clock, in pipes 2 and 3 at 11:15, and in pipe 4 at 11:30.

In Figure 16 are shown the depths of water at the five junctions and the computed width's, b, which affect many of the computations in the model. The values of b shown were computed with the depths in the pipes taken as the averages of the depths at the two junctions on either end of the pipe. During the periods of larger values of b, the computed potentials for bedload and suspended load transport were also larger, since $G_s = g_s b$ and $G_{ss} = g_{ss} b$.

Figure 17 shows the modeled deposition that occurred in the three upstream pipes and the solids discharged through the outfall. (The treatment rate for the primary treatment plant assumed at the end of pipe 3 was set at 16 cfs in this simulation, not 25 cfs as used in Case III.) The Figure shows that there was accumulation in all three upstream pipes but not in the outfall (pipe 4), which had the highest velocity during almost the entire period.

It is interesting to note that pipe 3, with the largest values of flow, velocity, depth, and width of the upstream pipes, also experienced the greatest amount and rate of deposition. This resulted because, as we will see in more detail below, there were more pounds of solids added to this pipe than to either of the two upstream, and also because the particles assumed in this exercise were all large enough and heavy enough to be removed in the 5,000 feet of pipe length. This occurred even though the early velocities in pipe 3 sometimes exceeded 2 feet per second and were 1.8 feet per second or above virtually throughout the period.

As soon as the hydrograph appeared, however, all three pipes were immediately scoured completely. The 3,000 pounds of accumulated solids in pipe 3 and all those in pipes 1 and 2 were gone within 1 hour (the period of reporting used in the model). Three hours later (hour 14) accumulation began again in pipe 1; the velocity had dropped in the 13th hour from 2.3 to 1.7

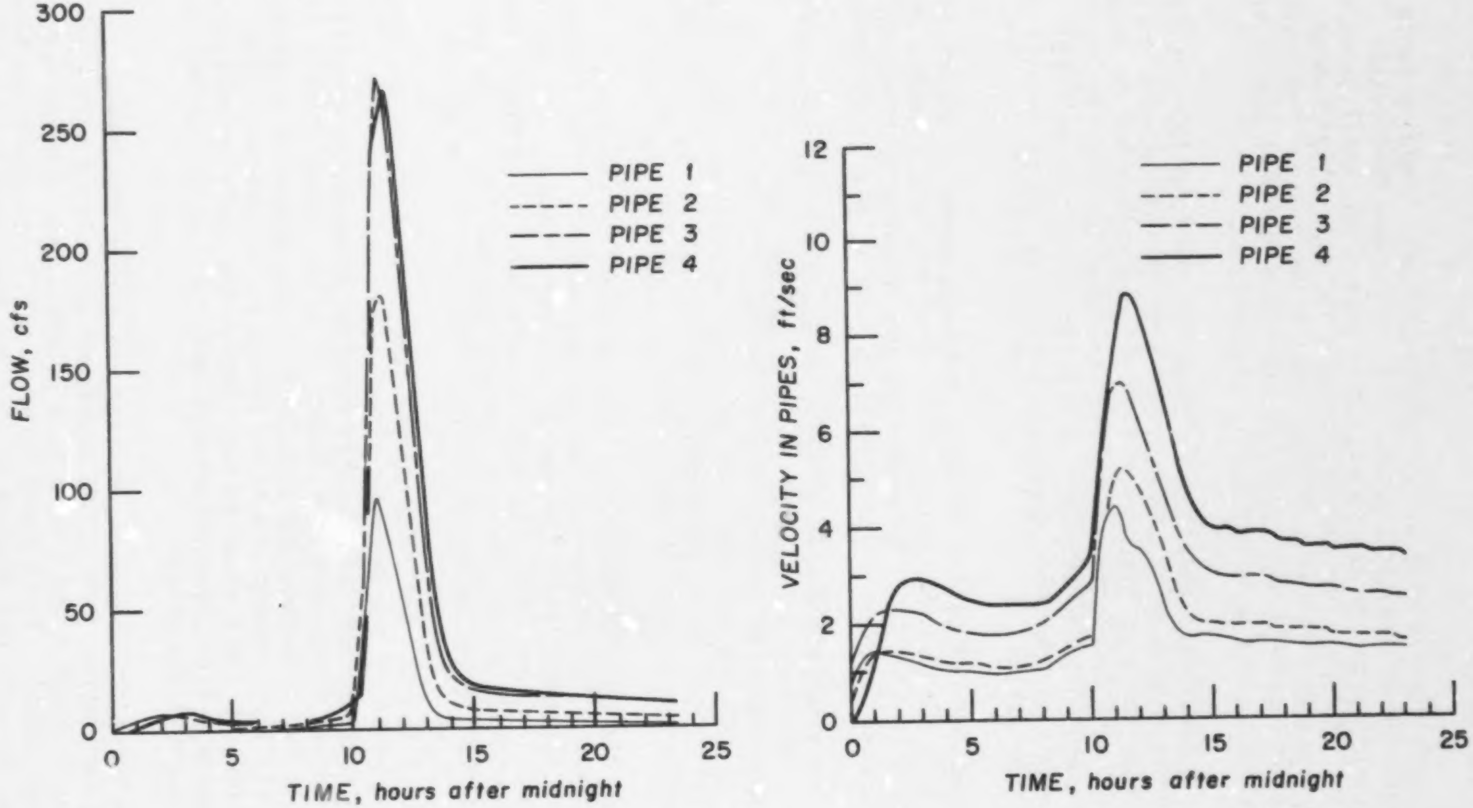


Figure 15
Flows and Velocities in Combined Sewer System During Delayed Storm Case

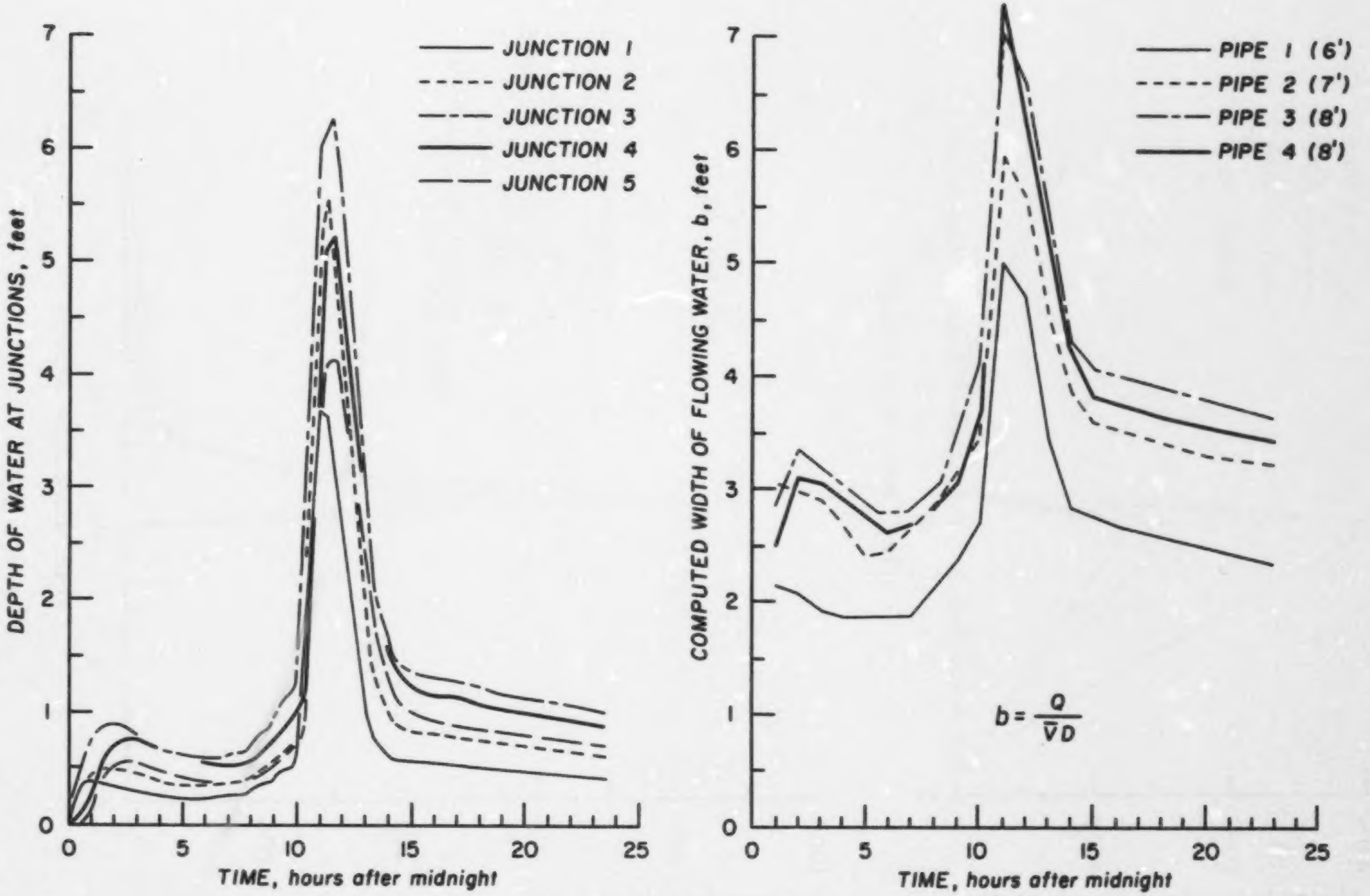


Figure 16
Depths and Widths in Combined Sewer System During Delayed Storm Case

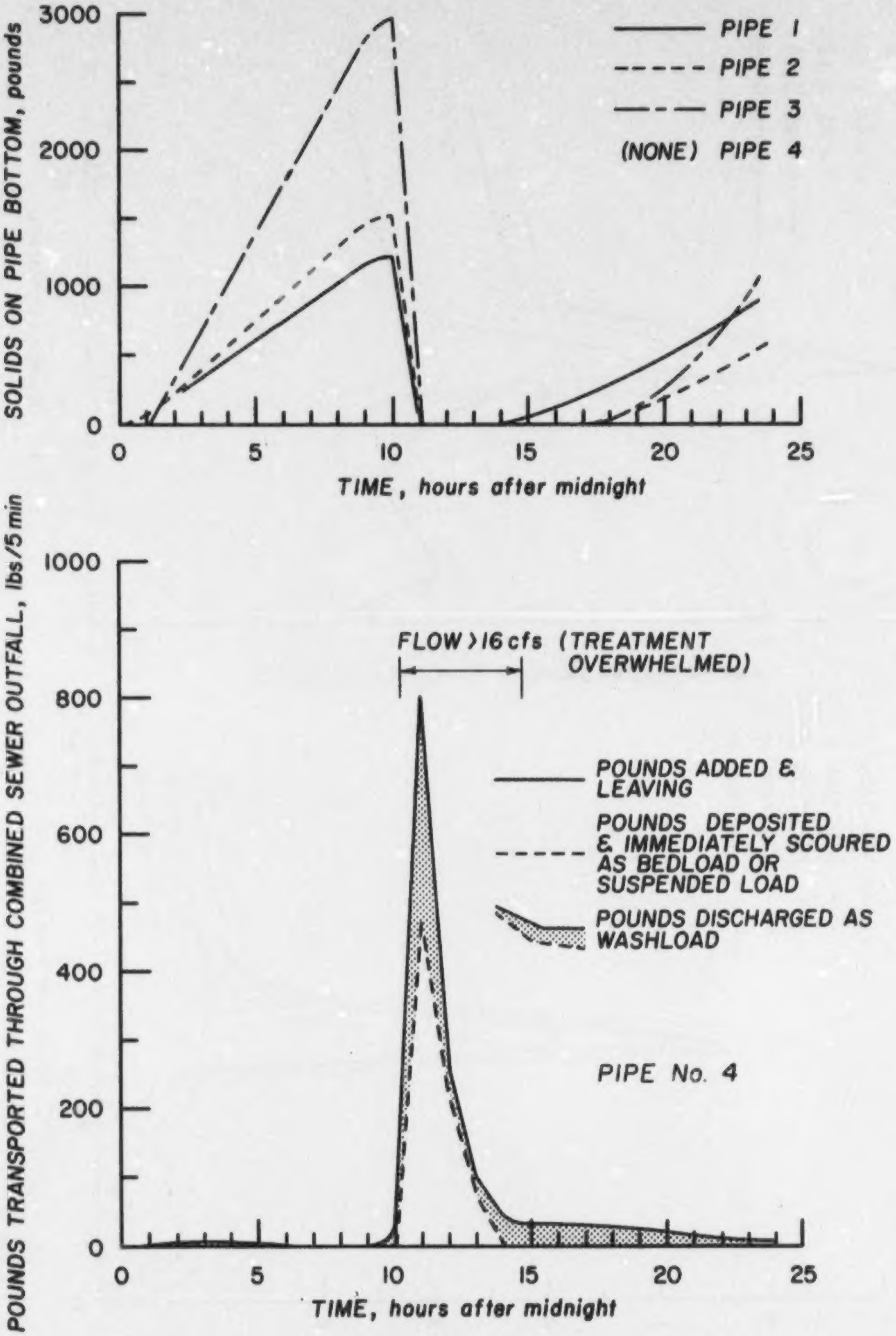


Figure 17
Disposition of Solids in Combined Sewer System During Delayed Storm Case

feet per second. In pipe 2 accumulation began at hour 16; velocity dropped in the 15th hour from 2.0 to 1.9 feet per second. But accumulation began again in pipe 3 at the 17th hour when velocity was still 2.9 feet per second; and accumulation continued at the highest rate of the three for the remainder of the period, while velocity never dropped below 2.5 feet per second.

The lower portion of Figure 17 shows what happened to these solids. Most of them left the sewer-treatment system through the outfall (pipe 4). The peak rate of solids transport was almost 800 pounds per 5 minutes (the time step used in the model) or 160 pounds per minute or 9,600 pounds per hour, which was reported at hour 11. The rate may have been higher earlier, between hour 10 and 11, though that could not be discerned since these results were printed at hourly intervals. Recall that the peak flow occurred in pipe 4 at 11:30, so the results do indicate that the peak load in the flush occurred prior to the peak flow--one of the characteristics of first flush.

The concentrations at the four junctions of interest are shown in Figure 18. Concentrations were reported every 15 minutes, so we can see that the peak concentrations occurred (were reported) at 10:15, just 15 minutes after runoff began. The concentrations were much lower at junction 4 both before and after the flush because the 16 cfs treatment plant was just upstream of that point. However, it cannot be missed that the concentration in the effluent, at junction 4, was the highest (790 mg/l) of the four during the storm, and 1 hour and 15 minutes before the peak flow.

Further details about the modeled solids behavior are given in a pipe-by-pipe, hour-by-hour account in Tables 8, 9, 10, and 11. Each table is for a single pipe. Table 8 shows the results for pipe 1. For most of the simulated period the dry-weather, sanitary inputs were 11.5-13 pounds every 5 minutes. In most hours all this was deposited but immediately scoured and moved along as bedload or suspended load. During the storm runoff period a small portion was transported as washload. It is worth noting that in the 10th hour more solids were scoured from the pipe bottom than were deposited. Also it is noteworthy that well after the storm there continued to be greater scour in this pipe than before the storm. This resulted because the flow and the velocity were higher in the later hours of the day due to the dry-weather inputs alone, which were shown in Figure 14. The velocities, shown in Figure 16, were about equivalent to what they were just before the storm and were about one-half a foot per second faster than they had been at the slowest flow in the morning. Moreover, from 2:00 p.m. forward, while scour was not equal to deposition, it was higher than it had been in the morning, and the velocity during this period was dropping from 1.7 to 1.4 feet per second.

Table 9 shows very similar results for pipe 2. More dry-weather inputs were being added to the upstream end of this pipe, in addition to the flow leaving pipe 1. Also during the storm another hydrograph was being added to junction 2 at its upstream end. Hence there were more pounds of solids and more flow added to this pipe. Also it was a foot larger in diameter than pipe 1. Still, behavior was very similar. There was net deposition before the storm; the solids that were scoured left the pipe, since there was no

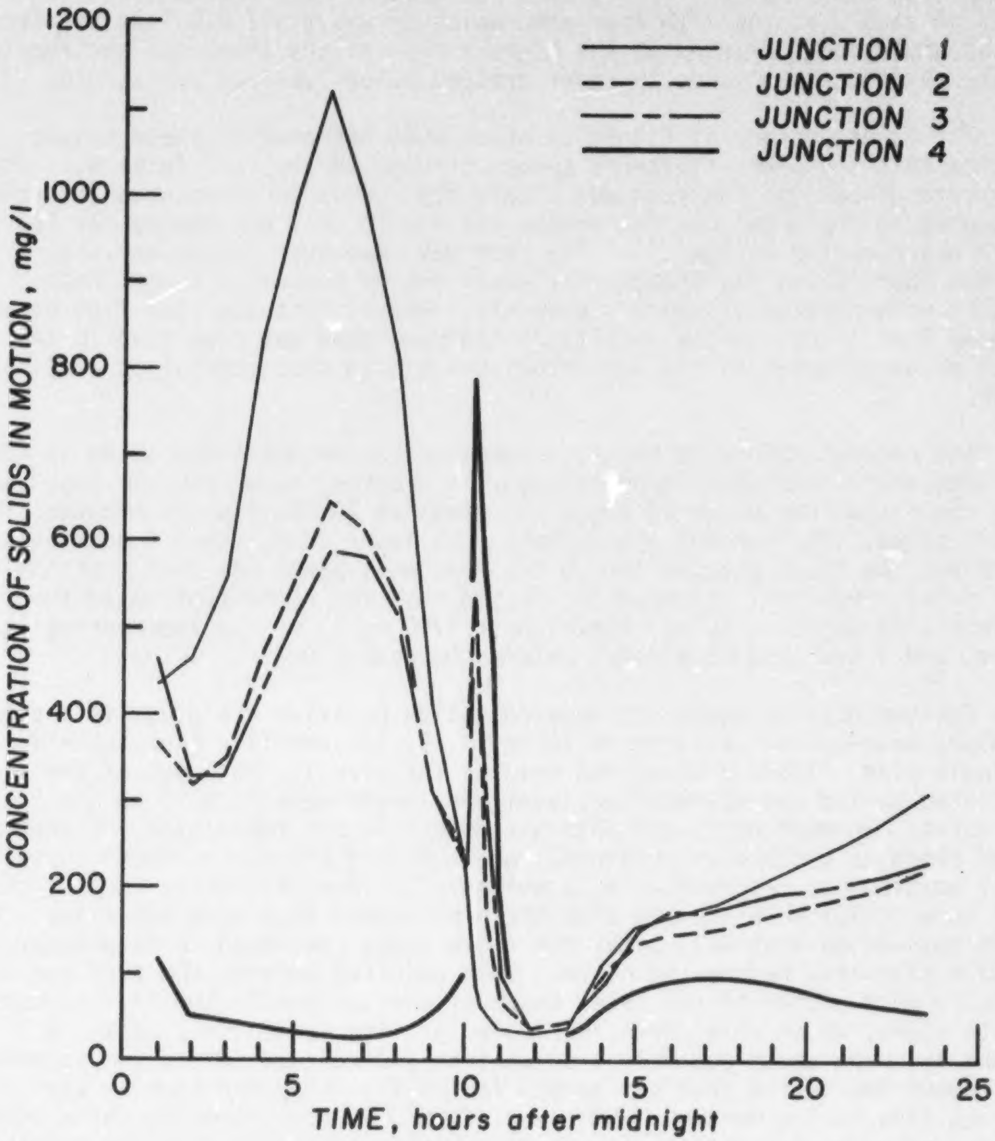


Figure 18
Concentrations of Solids Throughout Combined Sewer System
During Delayed Storm Case

TABLE 8. SOLIDS BEHAVIOR IN COMBINED SEWER--PIPE 1

	Pounds/5 minutes						
Time, hrs.	Added	Washed	Deposited	Scoured	Treated	Remaining	Leaving
5 min	4.5	4.5					4.5
1	11.6		11.6	0.9		73.9	0.9
2	11.8		11.8	1.3		199.9	1.3
3	11.7		11.7	0.6		330.8	0.6
4	11.5		11.5	0.1		464.9	0.1
5	11.8		11.8	0.1		603.2	0.1
6	11.8		11.8	0.1		743.2	0.1
7	12.2		12.2	0.1		888.0	0.1
8	12.5		12.5	0.1		1,034.9	0.1
9	13.0		13.0	2.5		1,177.8	2.5
10	12.5		12.5	13.4		1,219.9	13.4
11	127.5	0.7	126.8	126.8		0	127.5
12	64.8	0.2	64.6	64.6		0	64.8
13	17.9		17.9	17.9		0	17.9
14	11.7		11.7	9.1		5.5	9.1
15	12.0		12.0	6.2		64.3	6.2
16	12.1		12.1	5.6		138.0	5.6
17	12.0		12.0	4.9		219.9	4.9
18	12.0		12.0	4.3		309.2	4.3
19	12.0		12.0	3.7		404.8	3.7
20	12.0		12.0	3.2		507.8	3.2
21	12.0		12.0	2.9		615.4	2.9
22	11.9		11.9	2.7		725.9	2.7
23	11.9		11.9	2.5		837.9	2.5

TABLE 9. SOLIDS BEHAVIOR IN COMBINED SEWER--PIPE 2

	Pounds/5 minutes						
Time, hrs.	Added	Washed	Deposited	Scoured	Treated	Remaining	Leaving
5 min	4.7	4.7					4.7
1	15.2		15.2	2.7		79.1	2.7
2	18.9		18.9	5.3		233.1	5.3
3	17.9		17.9	4.4		395.2	4.4
4	16.7		16.7	2.3		564.1	2.3
5	16.7		16.7	1.9		739.7	1.9
6	16.8		16.8	1.3		920.8	1.3
7	17.5		17.5	2.1		1,107.1	2.1
8	18.0		18.0	2.2		1,292.8	2.2
9	20.5		20.5	8.4		1,466.8	8.4
10	28.8		28.8	21.3		1,516.3	21.3
11	300.0	3.1	296.9	296.9		0	300.0
12	146.2	1.8	144.4	144.4		0	146.2
13	48.7	0.3	48.4	48.4		0	48.7
14	26.7		26.7	26.7		0	26.7
15	23.2		23.2	23.0		0.2	23.0
16	23.1		23.1	22.0		7.0	22.2
17	22.5		22.5	20.7		24.0	20.7
18	21.8		21.8	18.4		57.5	18.4
19	21.1		21.1	16.0		110.6	16.0
20	20.6		20.6	14.1		181.0	14.1
21	20.2		20.2	12.2		269.6	12.2
22	20.0		20.0	10.7		374.6	10.7
23	19.8		19.8	9.4		492.8	9.4

TABLE 10. SOLIDS BEHAVIOR IN COMBINED SEWER--PIPE 3

	Pounds/5 minutes						
Time, hrs.	Added	Washed	Deposited	Scoured	Treated	Remaining	Leaving
5 min	3.9	3.9			0.6		3.3
1	27.2	21.6	5.6	0.1	18.4	5.5	3.3
2	43.6		43.6	14.8	8.9	359.2	5.9
3	39.6		39.6	12.4	7.6	689.2	4.8
4	35.9		35.9	7.6	4.8	1,026.9	2.8
5	35.1		35.1	5.9	3.7	1,371.6	2.2
6	34.9		34.9	3.7	2.4	1,736.7	1.3
7	36.0		36.0	4.2	2.7	2,114.4	1.5
8	37.6		37.6	6.8	4.3	2,492.6	2.5
9	43.4		43.4	15.8	9.4	2,813.6	6.4
10	59.7		59.7	66.8	36.7	2,953.2	30.1
11	590.6	17.2	573.4	573.4	18.2	0	572.4
12	246.6	5.6	241.0	241.0	7.0	0	239.6
13	97.9	4.4	93.5	93.5	7.5	0	90.4
14	58.1	1.1	57.0	57.0	15.9	0	42.2
15	55.2		55.2	55.2	24.1	0	31.1
16	56.2		56.2	56.0	24.6	0.6	31.4
17	55.3		55.3	54.3	24.0	9.3	30.3
18	53.0		53.0	48.0	21.8	47.4	26.2
19	50.7		50.7	40.8	19.1	144.3	21.7
20	48.9		48.9	35.9	17.3	285.2	18.6
21	47.0		47.0	31.2	15.4	462.0	15.8
22	45.3		45.3	27.1	13.8	668.9	13.3
23	43.9		43.9	23.5	12.4	903.9	11.1

TABLE 11. SOLIDS BEHAVIOR IN COMBINED SEWER--PIPE 4

	Pounds/5 minutes						
Time, hrs.	Added	Washed	Deposited	Scoured	Treated	Remaining	Leaving
5 min	0						0
1	0.8		0.8	0.1		1.6	0.1
2	4.9	4.9					4.9
3	5.4	5.4					5.4
4	3.4	3.4					3.4
5	2.4	2.4					2.4
6	1.7	1.7					1.7
7	1.5	1.5					1.5
8	2.0	2.0					2.0
9	5.2	5.2					5.2
10	21.7	21.7					21.7
11	797.9	320.1	477.8	477.8			797.9
12	258.5	31.8	226.7	226.7			258.5
13	100.3	25.8	74.5	74.5			100.3
14	42.9	42.9					42.9
15	31.1	31.1					31.1
16	30.8	30.8					30.8
17	30.4	30.4					30.4
18	27.3	27.3					27.3
19	22.8	22.8					22.8
20	19.5	19.5					19.5
21	16.5	16.5					16.5
22	14.0	14.0					14.0
23	11.7	11.7					11.7

treatment; scour was greater after the storm than before; and washload transport occurred only in the high-flow, high-velocity conditions during the peak runoff period.

Table 10 shows the results for pipe 3. More sanitary waste inputs and another hydrograph were added at the upstream end of this pipe (junction 3). So there were more pounds being added to this pipe than to the two upstream. While flow and velocity were higher in this pipe than in the previous two, there was still net deposition occurring; although in the 10th hour there was a net scouring. Also for this pipe there was some removal by treatment. Still, many of the characteristics were the same as for the previous two pipes--most of the solids entering the pipe were able to deposit, partially removed immediately as bedload or suspended load transport; washload occurred only at the highest and fastest flows; and deposition with net accumulation began again after the storm had passed.

Unfortunately, the model was not directed to print the amounts of material moving in each period as washload, suspended load, and bedload individually. (The washload numbers in the Tables were computed by difference.) But the model did print the amounts of material removed by the treatment plant in these three categories for the entire simulated period. These were: washload, 369 pounds; suspended load, 2,071 pounds; and bedload, 2,269 pounds. Since the removal percentages specified were 85, 65, and 35 percent, respectively, the total amounts in motion in these three species in pipe 3 can be computed. These were: washload, 434 pounds; suspended load, 3,186 pounds; and bedload, 6,483 pounds. So most of the material in motion during the entire 23.5 hour period moved as bedload, at least in this pipe.

Table 11 gives the results for pipe 4, the outfall from the system. This pipe was steeper than the others, and velocities here were above 2.5 feet per second virtually all the time. Moreover, the depths were lower; and there was slight chance for accumulation. Hence most of the time the material moved through the pipe was washload and no accumulation occurred. Curiously, though, there was apparent deposition with immediate scour during the period of greatest flow and velocity. How could that be?

The answer lies in the definition of critical velocity for deposit-free flow, V_c (see [16], p. 19). During the early part of the simulated period, the critical velocity was roughly:

$$V_c = 0.9 \, [2 \times 32.2 \times 0.5 \times (1.2 - 1)]^{1/2} = 2.3 \text{ ft/sec}$$

while the velocity in the pipe was 2.4 to 3.0 feet per second. So no deposition could occur.

In the storm runoff period the critical velocity was higher because the depth was greater and the weighted specific gravity of the particles was greater:

$$V_c = 0.9 \, [2 \times 32.2 \times 4.5 \times (1.4 - 1)]^{1/2} = 9.69 \text{ ft/sec}$$

However, the maximum velocity attained in this pipe was 8.8 feet per second, so some deposition was possible. It is also possible, of course, that the critical velocity definition, particularly the coefficient of 0.9, is not correct. This was discussed earlier. But the coefficient would have to be below 0.2 for the critical velocity to be 2.0 feet per second, and the coefficient would have to be below 0.3 to yield a critical velocity of 3.0 feet per second, with this depth and specific gravity. The lowest value reported for the coefficient is 0.7. So the implication is fairly clear: Deposition can occur under conditions where the velocity is substantially above 2.0 feet per second, the long-established rule-of-thumb criterion for self-cleansing. It is interesting to note from all these results, however, that when velocities increased past 2.0 feet per second there was net scour of deposited material. So perhaps the old criterion is not bad for a rule-of-thumb scour criterion (for noncohesive materials); but this should not be confused with a no-deposit criterion. In summary, even if the model is not correct in every detail, it gives strong evidence that particles may be deposited and immediately moved along as bedload or suspended load in the same time period and that a velocity sufficient to move particles can be much smaller than the velocity necessary to keep them fully suspended. This is consistent with other findings (Graf, 1971; ASCE, 1975).

To complete the discussion of critical velocity in pipe 4, it can also be noted that after the runoff period the depth lowered to about 1.0 foot and below and the specific gravity of the particles lowered again to 1.2. So the critical velocity was then not greater than:

$$V_c = 0.9\ [2 \times 32.2 \times 1 \times (1.2 - 1)]^{1/2} = 3.2 \text{ ft/sec}$$

The velocity in pipe 4 ranged from 4.8 down to 3.3 feet per second after the hydrograph passed, so again no deposit occurred and all the load was transported as washload.

This simulation alone, analyzed in some detail, indicated that the model was functioning or executing as had been intended and it produced answers that could be explained both in terms of behavior noted generally in the past and in terms of the expressions programmed into it. It cannot be judged that the expressions used, such as that for the critical velocity or even Kalinske's expression for bedload transport are absolute descriptions of truth, but the results indicate that the model with these expressions in it can simulate or approximate prototype behavior adequately. The results from other simulation cases for the example problem are summarized below.

OTHER RESULTS

The Effect of Conduit Slope

Attention is now turned to the other example cases wherein the hydrographs were added to the upstream junctions just after the simulation began (at midnight). The first three cases were for the combined sewer system with a 25 cfs primary treatment plant at the downstream end of pipe 3. The differences among the three cases were in pipe slope only. The slopes in

Case I varied from 0.0008 foot per foot in pipe 3 to 0.002 foot per foot in pipe 4. (See the junction elevations in Table 6.) Case II gave each pipe a 50 percent greater slope; and Case III was the intermediate condition, wherein slopes were increased only 25 percent over the Case I conditions.

The flows and velocities in pipe 2, upstream of the treatment plant, are shown for Cases I and II in Figure 19. The values for Case III, the intermediate slope case fall in between those shown and are omitted from the figure for clarity. It can be seen that the more steeply sloped pipes gave rise to very slightly higher flows and somewhat higher velocities, as would be expected.

The resulting concentrations of solids in motion ("suspended" solids) are shown in Figure 20. The results shown are for junctions 3 and 4. Junction 3 was at the downstream end of pipe 3 just following the treatment plant. The results show that the steeper slopes led to higher concentrations, both before and following treatment. Figure 21 shows rather dramatically why that occurred. Here are shown the pounds of accumulated solids in pipe 2. The lower slope case led to substantial accumulation in the upstream pipes; while in the highest slope case, the dry-weather flows were able to rescour the pipes in the afternoon and early evening hours. Therefore, the concentrations were lower downstream in the low slope case because so few solids were reaching the downstream junctions. In another situation where slope is always kept sufficiently steep to keep solids in motion toward a plant, solids concentrations downstream would be minimized by increasing slope until the treatment capacity of the plant was just attained by the resulting increase in flow rate in the sewers. In other words, increasing pipe slope will not always lead to increased solids concentrations and indeed may lead to decreased concentrations. The determinant is the degree to which solids accumulation can occur upstream.

For the cases analyzed, the increased slopes did allow more solids to reach the treatment plant so they could be removed. In Case I the treatment plant removed 4,180 pounds of solids during the event. In Case II, the steepest slope case, the removal was 7,068 pounds. In the intermediate slope example, Case III, the plant removed 5,551 pounds. So while the plant removed 25.6 percent of the solids entering the system during the event in the steep slope case, 63.3 percent of the entering solids were discharged, while 11.1 percent accumulated on the pipe bottoms. The flattest slope pipes led to only 15.1 percent removal by the plant, and to only 49.8 percent of the solids being discharged, but 35.1 percent of the entering solids remained behind on the pipe bottoms.

The Effect of Conduit Shape

The simulation of cunette-shaped cross-sections produced some erratic results. But they are worth reviewing. The flows and velocities in a rectangular combined sewer system with cunettes are compared with those for a circular combined sewer system in Figure 22. The conditions being analyzed are similar to the Case III conditions before--the mildly sloped, combined sewer with a treatment plant at the end of pipe 3. In both these examples, however, the treatment capacity of the primary plant was 16 cfs.

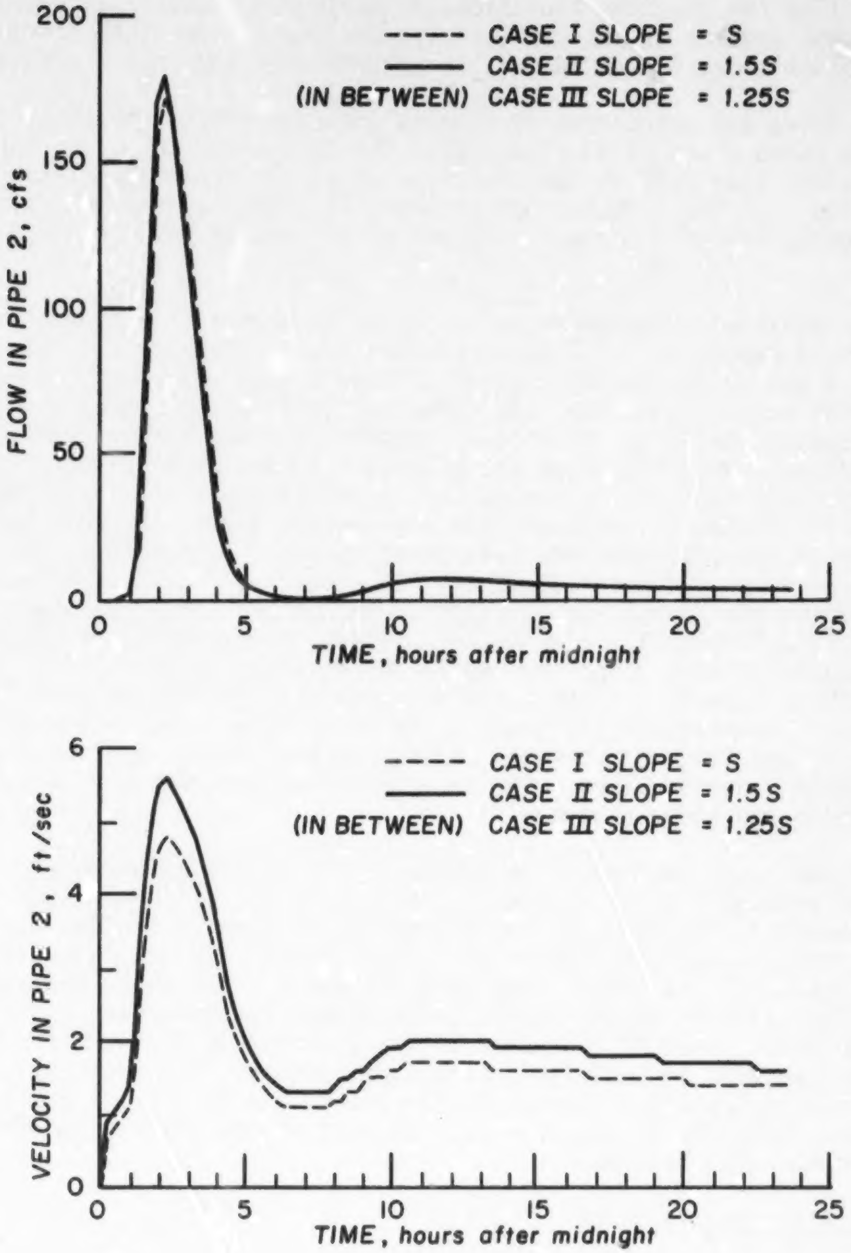


Figure 19
Flows and Velocities in Pipe 2 of Combined
Sewer Systems with Different Slopes

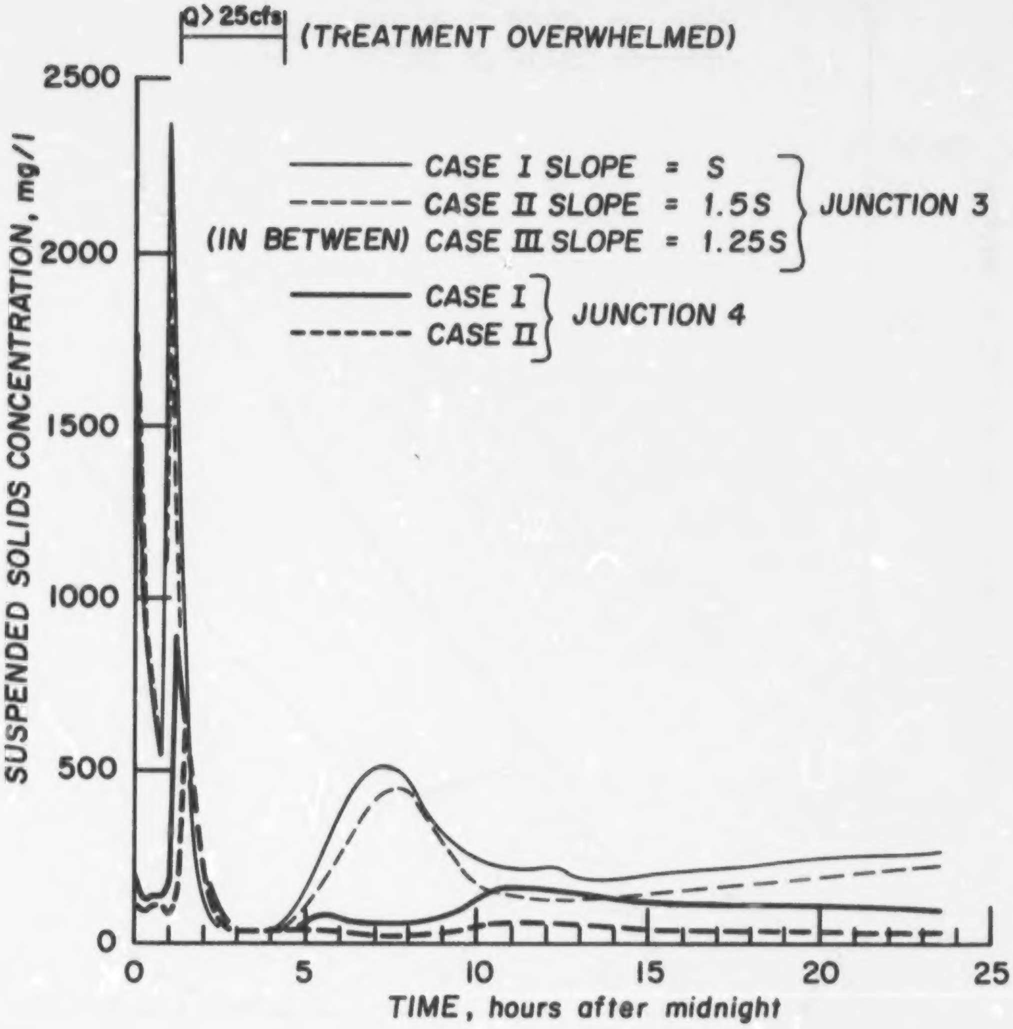


Figure 20
Downstream Solids Concentrations in Combined Sewer Systems as Functions of Pipe Slopes

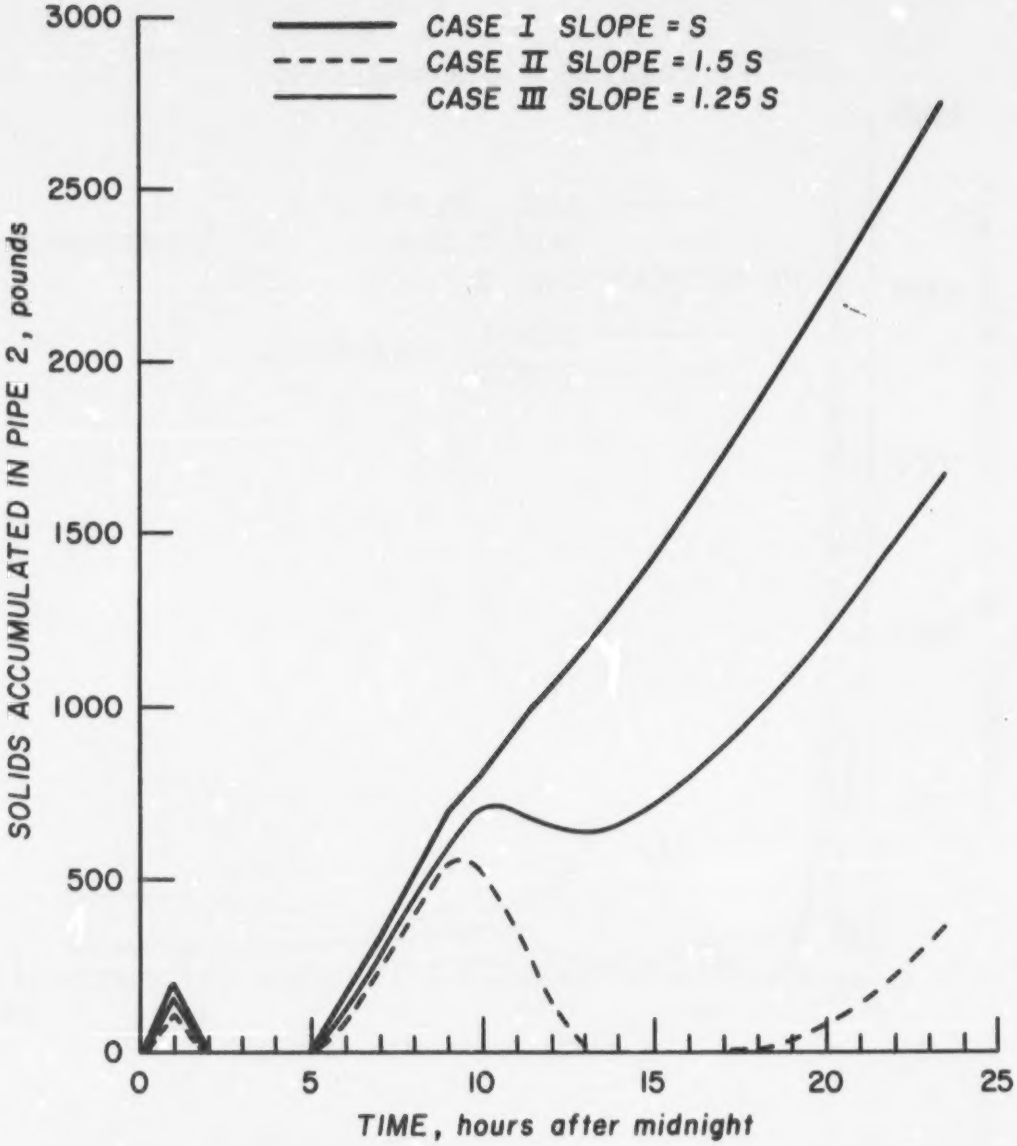


Figure 21
Upstream Accumulation of Solids in Combined Sewer
System as a Function of Pipe Slope

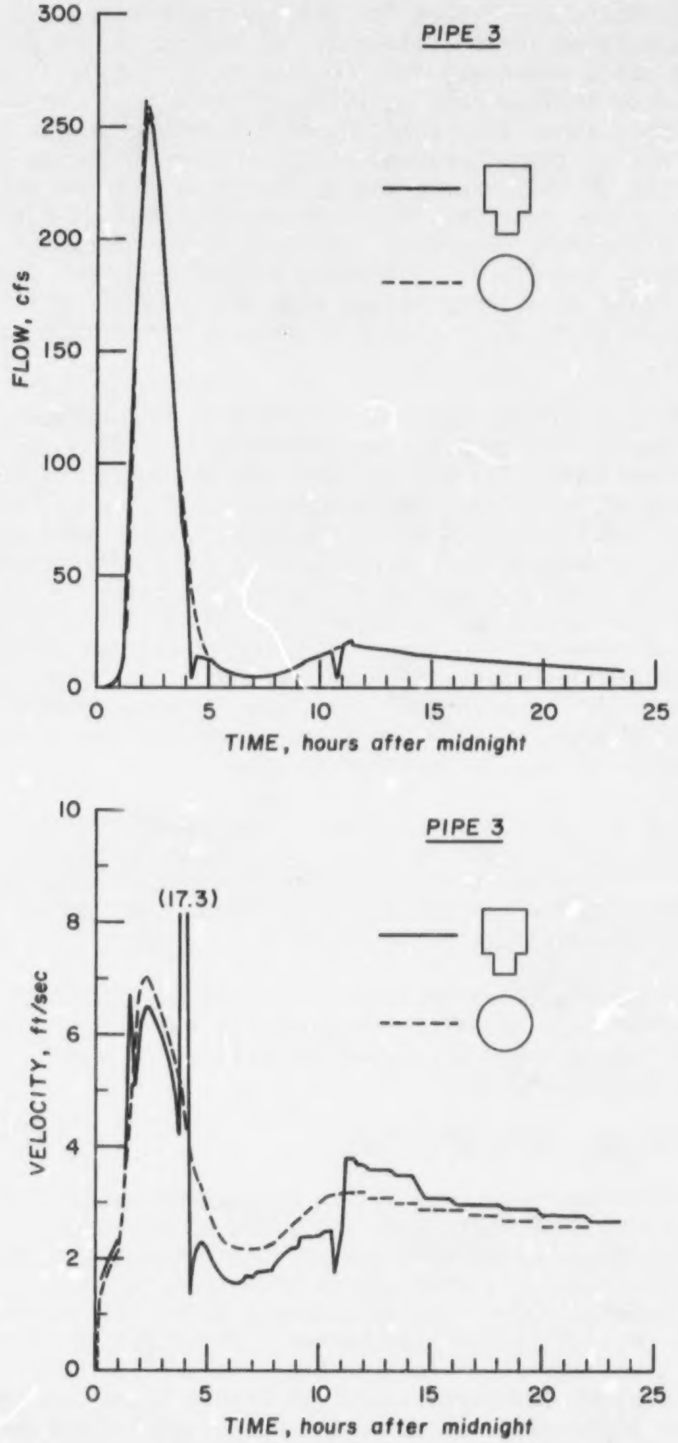


Figure 22
Flows and Velocities in Combined Sewers of Different Shapes

The flows shown in the figure for the two cases were almost identical throughout the simulated period, although two incongruities occurred in the hydrograph. One was a momentary drop in flow rate at time 4:15, and the other was a momentary drop in flow rate at 10:45. Two other incongruities appear in the velocity data shown at the bottom of Figure 22, one at 1:45 and the other at 4:00. All of these occurred at times when the depth of water at one end of the pipe was in the cunette and at the other end the water was deep enough to be out of the cunette. This caused the model to have widely different areas and widths to average for conditions at the center of the pipe and this led to the discontinuities. Nonetheless, the model was able to recover from these conditions in one time step and to go forward to represent the majority of the period with results comparable to those for the circular pipe.

In Figure 23, it can be seen that the Quality Model results were quite similar for the two cases, despite the momentary upsets in the hydraulics results for the cunettes. The figure shows the treatment rate for solids removal at the end of pipe 3 and the accumulation of solids in all the conduits. It is interesting, if quite unexpected, that the circular pipe system afforded more removal by treatment and less accumulation than did the cunette system. This was a function of the sizes chosen for the examples and is not a general impugnation of cunette's reputed self-cleansing properties. We simply made the cunettes too large. The dimensions given to pipe 3 were a 7-by-7 foot box with a 2.8-by-2.8 foot cunette in the bottom. The cunette contained all the flow after the 4th hour; and at these generally flat slopes, the potential for deposition with immediate scour was not as great in this flat-bottomed box as it was in the rounded pipe.

Because there was more accumulation in the cunette system, though, there was also marginally less discharge of solids from this system. Figure 24 shows the rate of solids discharge through the outfall, pipe 4. The discontinuities in the results for the rectangular conduit occurred as a result of the flow and velocity discontinuities upstream.

Figure 25 shows the concentrations of solids as discharged into the outfall from the two pipe systems. The behavior was almost identical, although the rectangular-cunette shaped sewer produced a very slightly lower and earlier peak concentration.

The Effect of Treatment Types and Sizes

Figure 26 shows the suspended solids concentrations at the outfall resulting from adding different types and sizes of treatment to the combined sewer system. All these cases were analyzed with the highest slopes and for circular pipes. The only differences were the treatment types, sizes, and locations. Several things are of interest here. First, the swirl concentrators upstream were of little assistance in this case in reducing downstream concentrations at the outfall, although they did provide some improvement over primary treatment alone downstream. But in Figure 27 we can see that much less accumulation occurred in the pipes when the swirl concentrators were in place. After hour 13 virtually all the accumulation in the case with the swirl concentrators occurred in pipe 1, which was upstream of the first

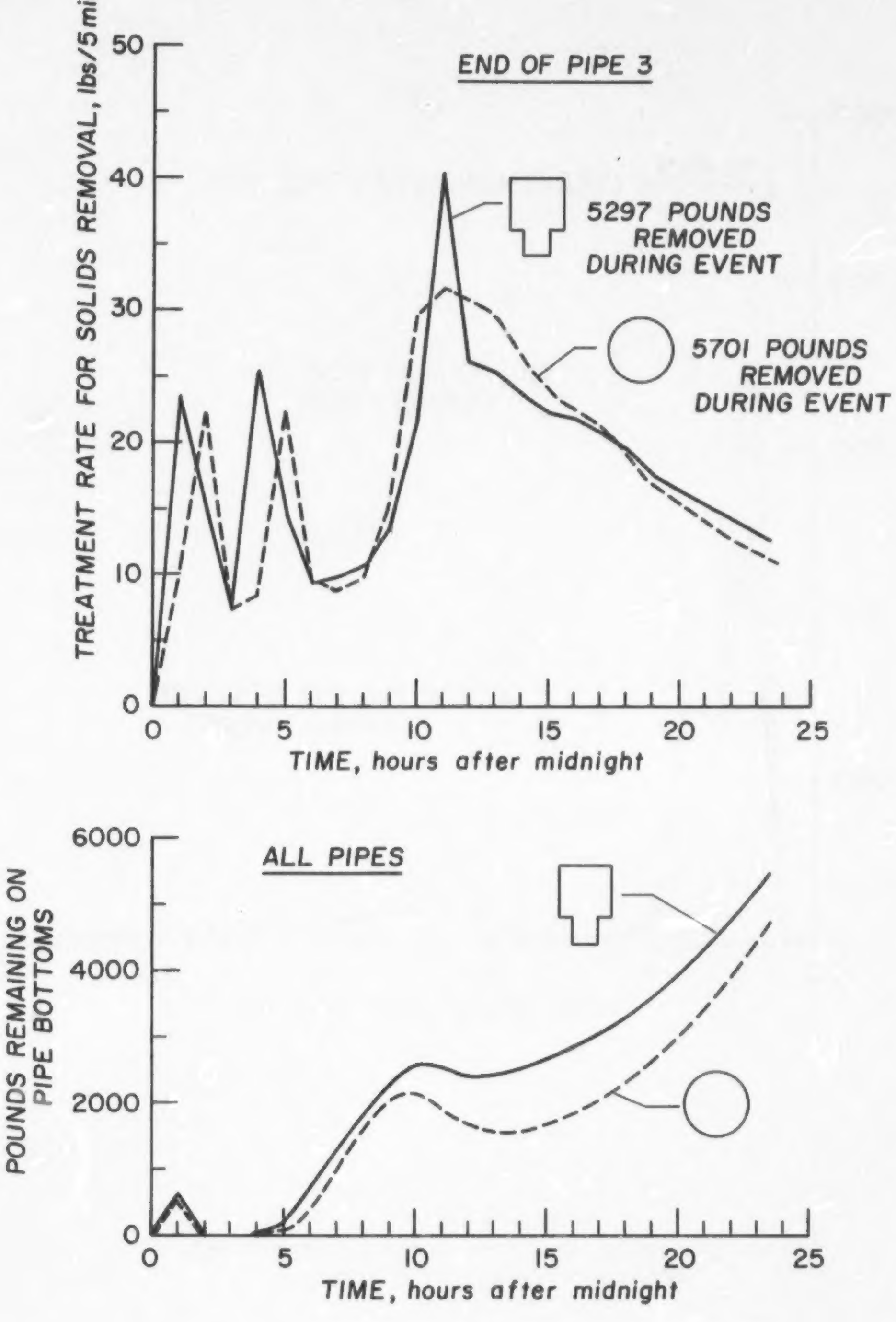


Figure 23
Disposition of Solids in Combined Sewer Systems
Having Different Pipe Shapes

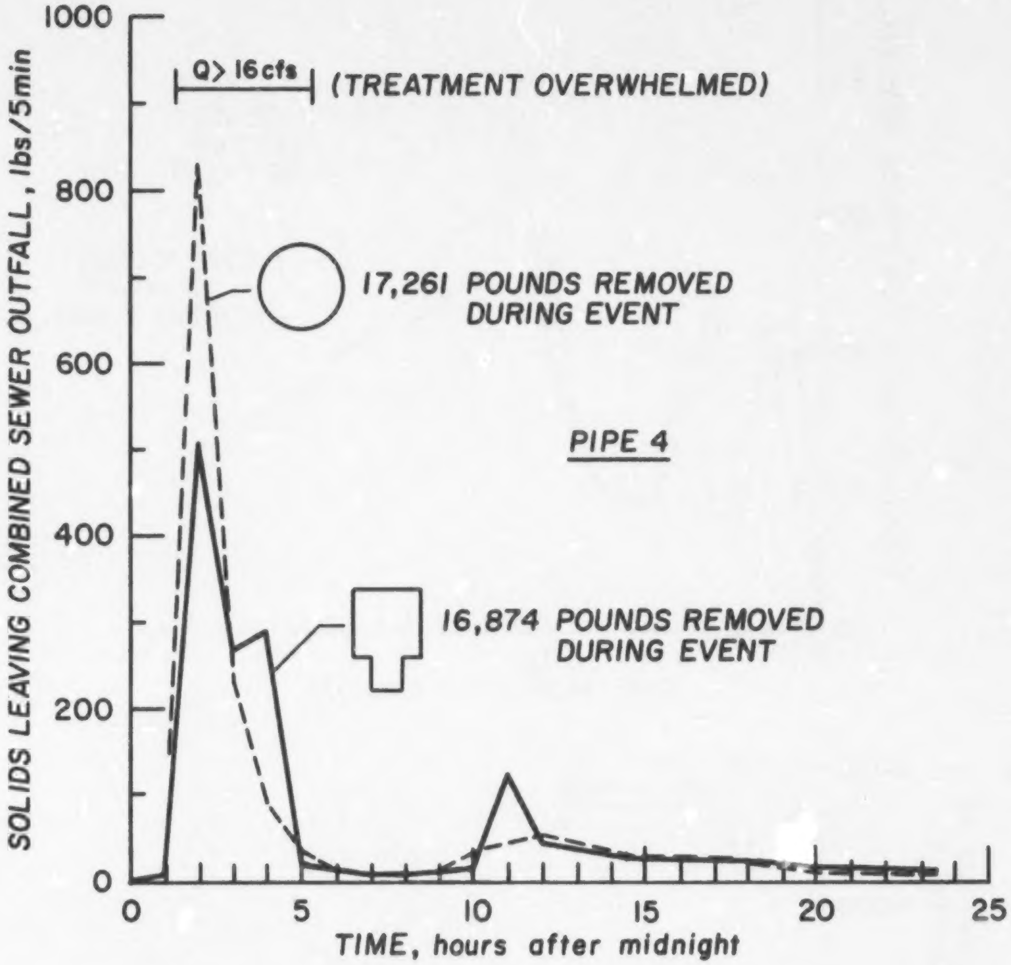


Figure 24
Solids Discharge Rate from Combined Sewer System
Having Different Pipe Shapes

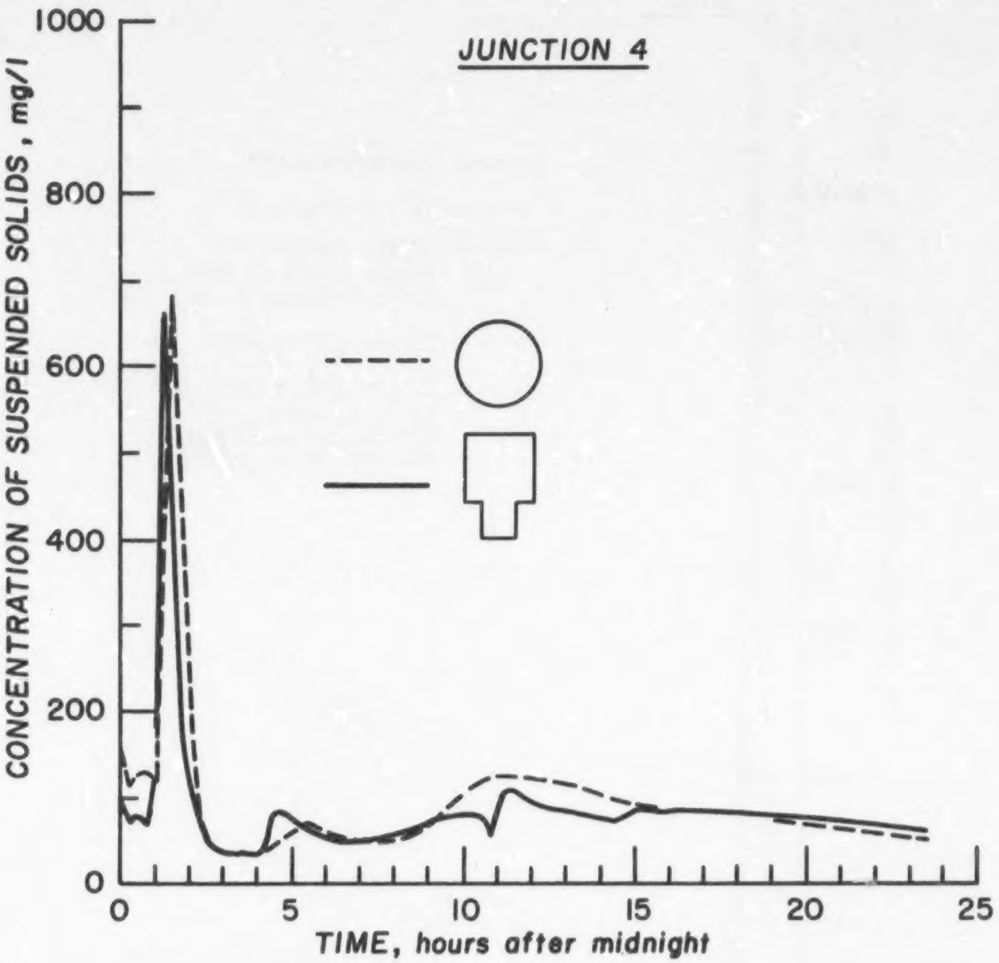


Figure 25
Concentration of Solids in Combined Sewer Outfalls
for Systems Having Different Pipe Shapes

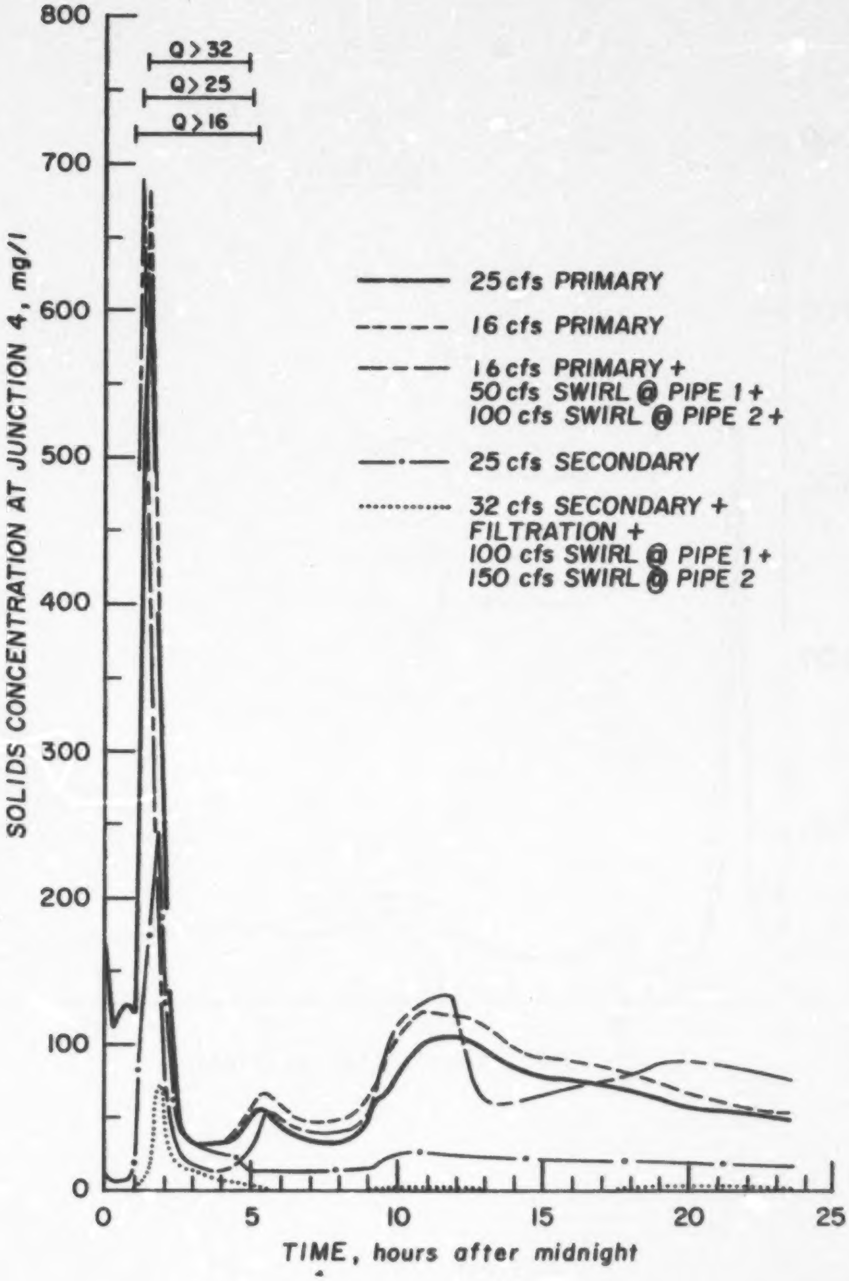


Figure 26
Effects of Treatment Type and Size on Effluent
Solids Concentration

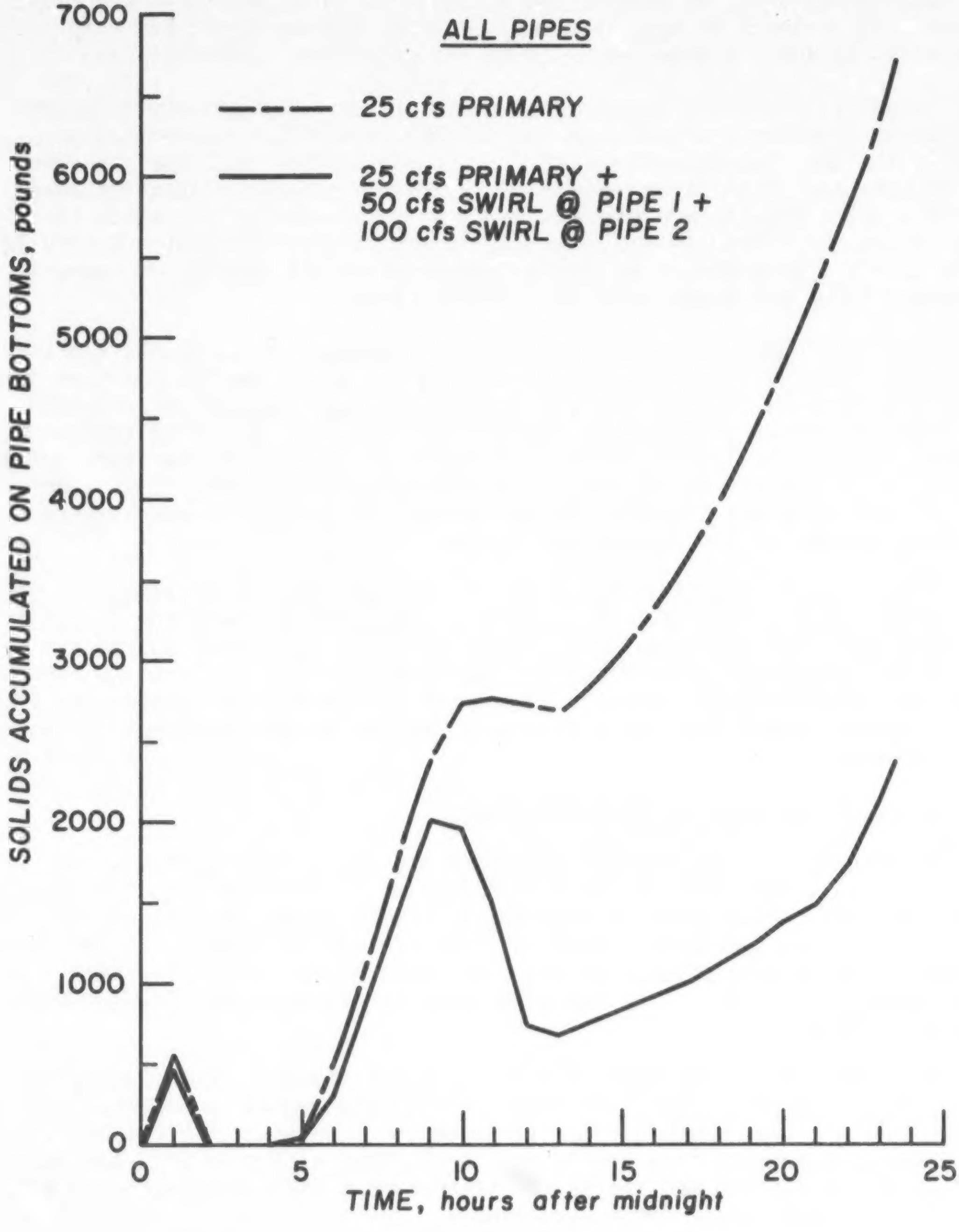


Figure 27
Effects of Upstream Stormwater Treatment in
Combined Sewer System on Accumulation of Solids

device. In the case with only the primary plant at pipe 3, most of the accumulation occurred in pipes 2 and 3, still ahead of the first treatment device. So in terms of mass removed, the swirl concentrators did help substantially and the model was able to indicate their contribution.

Secondly, providing secondary treatment rather than primary treatment downstream produced a significant improvement in effluent concentration. Finally the most improvement resulted by providing both upstream and downstream treatment with rather complete treatment including filtration downstream. These results are quite case specific, of course; and again the results are reflective of the accumulation and later scour that occurred in these fairly flat sewers. In another sewer system the results of comparing treatment types and sizes could be quite different.

Figure 28 shows the effects of various treatment types on the sanitary part of the separate system. The concentrations shown are for junction 40 which follows pipe 30 where the dry-weather treatment plants were located. The expected result of improved removal with increased degree of treatment ensued. Notice there is no early peak of solids related to the storm hydrographs, since the stormwater was all in the other storm sewer. Also, no infiltration flow was assumed. So the pattern of concentrations is very much the same as that of the dry-weather inputs.

These results show that the model is able to simulate treatment to obtain expectable results, but also it is able to show that accumulation of solids upstream and later resuspension of those particles may rather substantially reduce the effectiveness of treatment for reducing final effluent concentrations. It certainly shows that the sewer system must be designed to keep solids moving toward downstream treatment devices or the treatment can be largely wasted.

The Effect of Combined vs. Separate Sewers

In establishing the conduit sizes for all the example problems, we decided to make the storm sewers 1 foot smaller in diameter than the combined sewer sections, which were analyzed first. As it turned out, almost all the total flow during the storm period was comprised of the runoff, so the storm sewers needed to be virtually as large as the combined sewers had been. In other words, surcharge of the upstream storm sewers occurred, because they were a little too small.

But, even though the model treats surcharge somewhat capriciously (it sets the flow equal to full-pipe flow and the depth equal to diameter; and it does not compute a pressurized flow rate); nonetheless, the results are presented here because they are instructive. They will show what the model does in this situation and they show, if perhaps a bit inaccurately, what happens in a sewer under these conditions.

Figure 29 compares the combined sewer flows and velocities of Case III with the storm and sanitary sewer flows of Case IX, which is the counterpart to Case III in that the slopes were intermediate and all pipes were circular. This Figure shows the results for the second pipe from the upstream end of

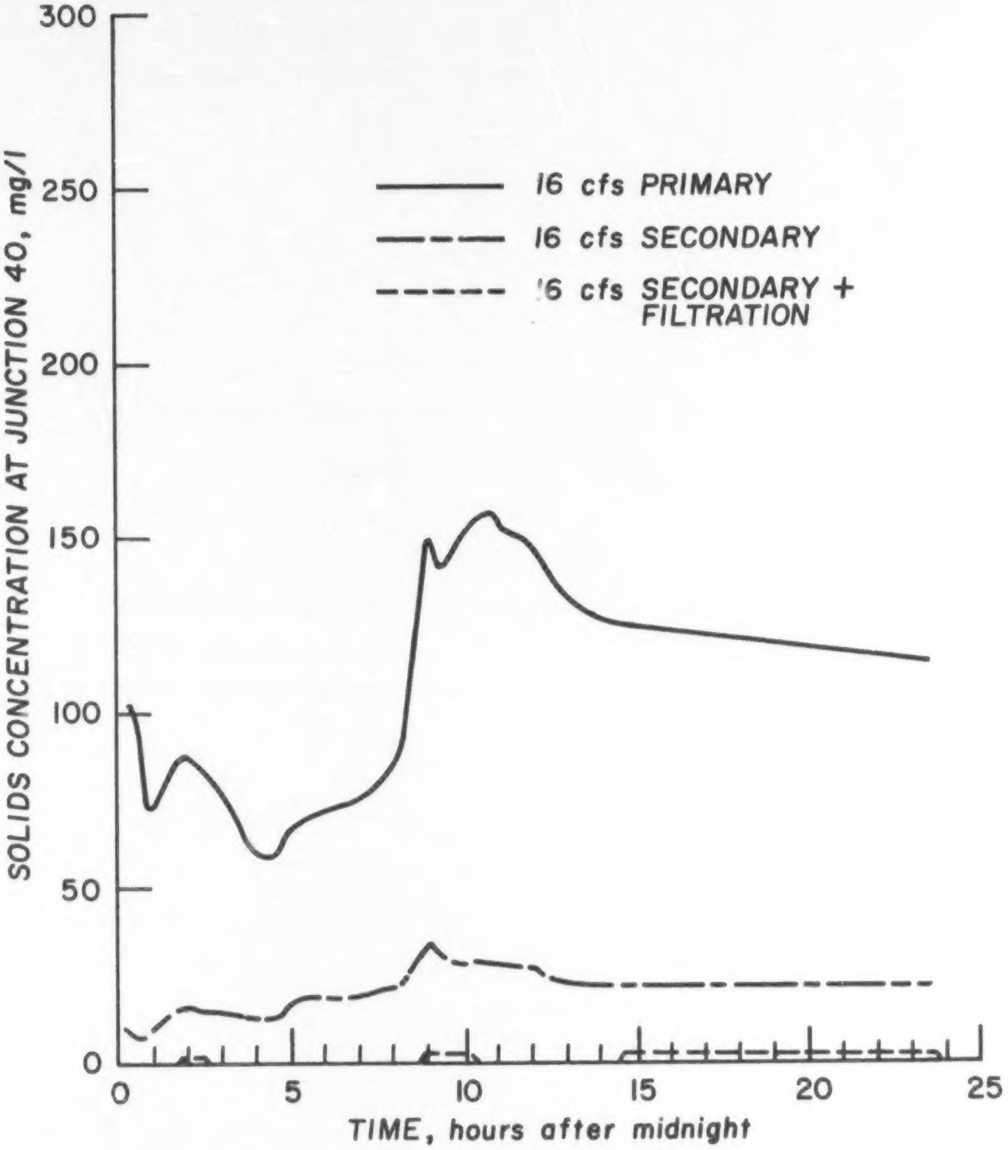


Figure 28
Effects of Treatment Type on Effluent Solids Concentrations from Sanitary Sewer System

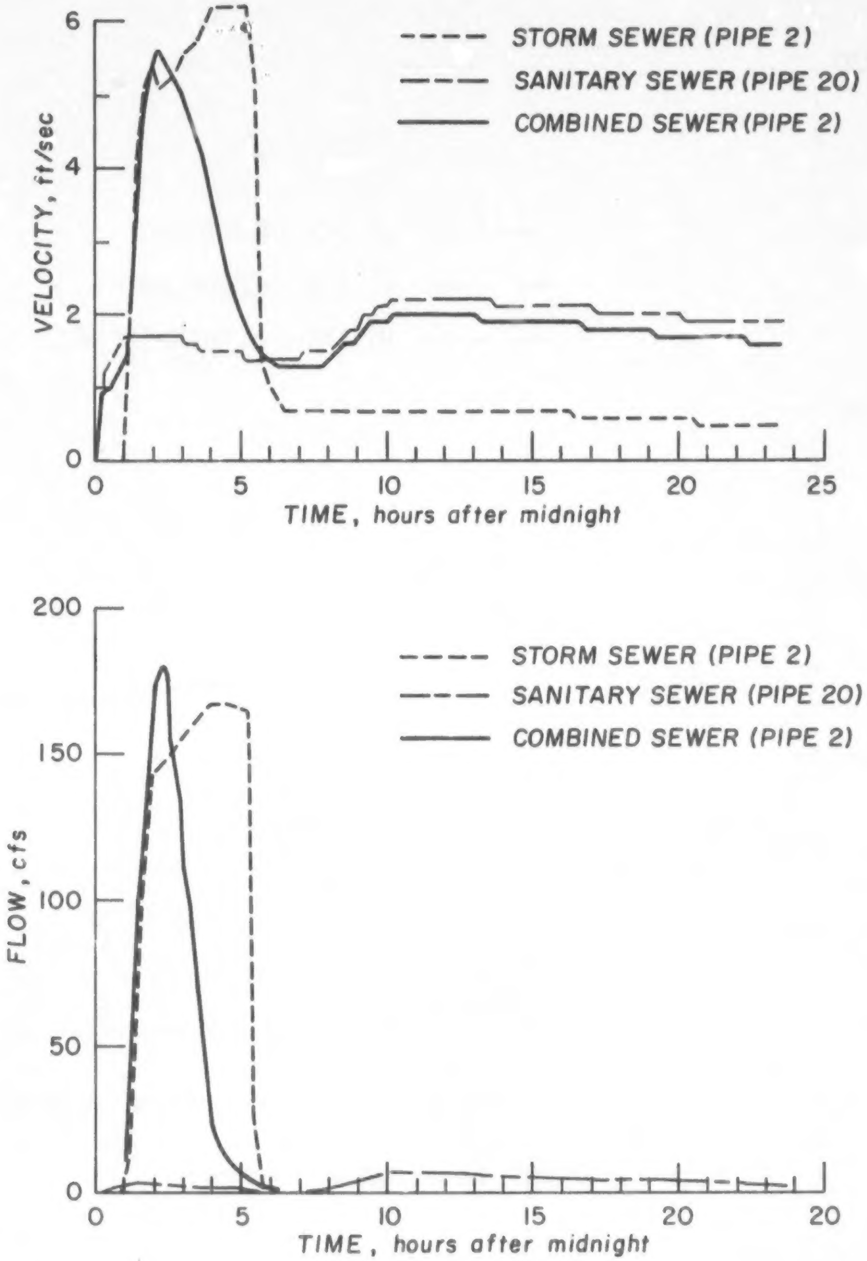


Figure 29
Velocities and Flows in Upstream Storm, Sanitary, and Combined Sewers as Simulated

all three pipe networks--pipe 2 of the combined system, pipe 2 of the storm sewer, and pipe 20 of the sanitary sewer. The curves show that the surcharge of the upstream storm sewer kept both flows and velocities high for a longer period than occurred in the combined sewer. They also show that while the sanitary flows in pipe 20 were fairly small by comparison to the storm flows, the velocities remained more uniform than in either of the storm-carrying pipes.

Figure 30 presents the flows and velocities in the downstream outfalls--pipe 4 in the combined system, pipe 4 in the storm sewer, and pipe 40 in the sanitary sewer. While the storm sewer flows and velocities remained high for an extended period, pipe 40 was not itself under surcharge, i.e., this pipe was never full. But its flow pattern and hence its velocity were controlled by the flows from upstream. Hence the flows and velocities in the storm sewer outfall appear quite different from those in the combined sewer outfall. The flows and velocities in the sanitary sewer outfall were again relatively uniform throughout the day.

Figure 31 shows the solids accumulation consequences in the various sewer types upstream. The top curve, i.e., the most accumulation occurred for the sanitary sewer. Accumulation during the first nine hours was fairly uniform, this being the same period when the velocity in the sanitary sewer was less than 2.0 feet per second. From hour 10 to hour 17, while velocity was greater than 2.0 feet per second, there was a net scouring of accumulated solids. Following that, as velocity dropped to 2.0 feet per second and below, there was a net accumulation again. It is interesting that when velocity was exactly 2.0 feet per second, the amount of scour and the amount of accumulation were virtually zero. It is worth reiterating that the model does not use the 2.0 feet per second criterion for self-cleansing in any explicit way. Nonetheless, values very near this velocity often resulted when the net accumulation rate was zero. Note for example that the net accumulation rate was zero for the combined sewer at hour 9, when the velocity there was 1.9 feet per second. While the velocity was at 2.0 feet per second, between hours 10 and 13, net scouring occurred in that sewer. When velocity fell to 1.8 feet per second and below, accumulation began again in the combined sewer too. Results are given in the Figure for both the combined sewer having the swirl concentrator at pipe 1 and for the combined sewer without one. It can be seen that the concentrator had little effect on the accumulation in pipe 2 of the combined sewer. This results in part because many of the solids in pipe 1 were accumulating there also and not reaching the concentrator for removal.

Accumulation in the storm sewer does not show in the Figure at all, even though after hour 6 the velocity was substantially below 2.0 feet per second. This results because the stormwater contained very few solids after the initial runoff period, which was sufficient in velocity in the beginning to keep the pipe scoured clean. After hour 6 there was in fact a minute amount of deposition in the storm sewer; but this amounted to only 1.7 pounds after 23.5 hours, which cannot even be plotted on the graph at the scale used in Figure 31.

Figure 32 shows the final result of using the sanitary sewer system with various levels of treatment. These are the results for the entrance to the

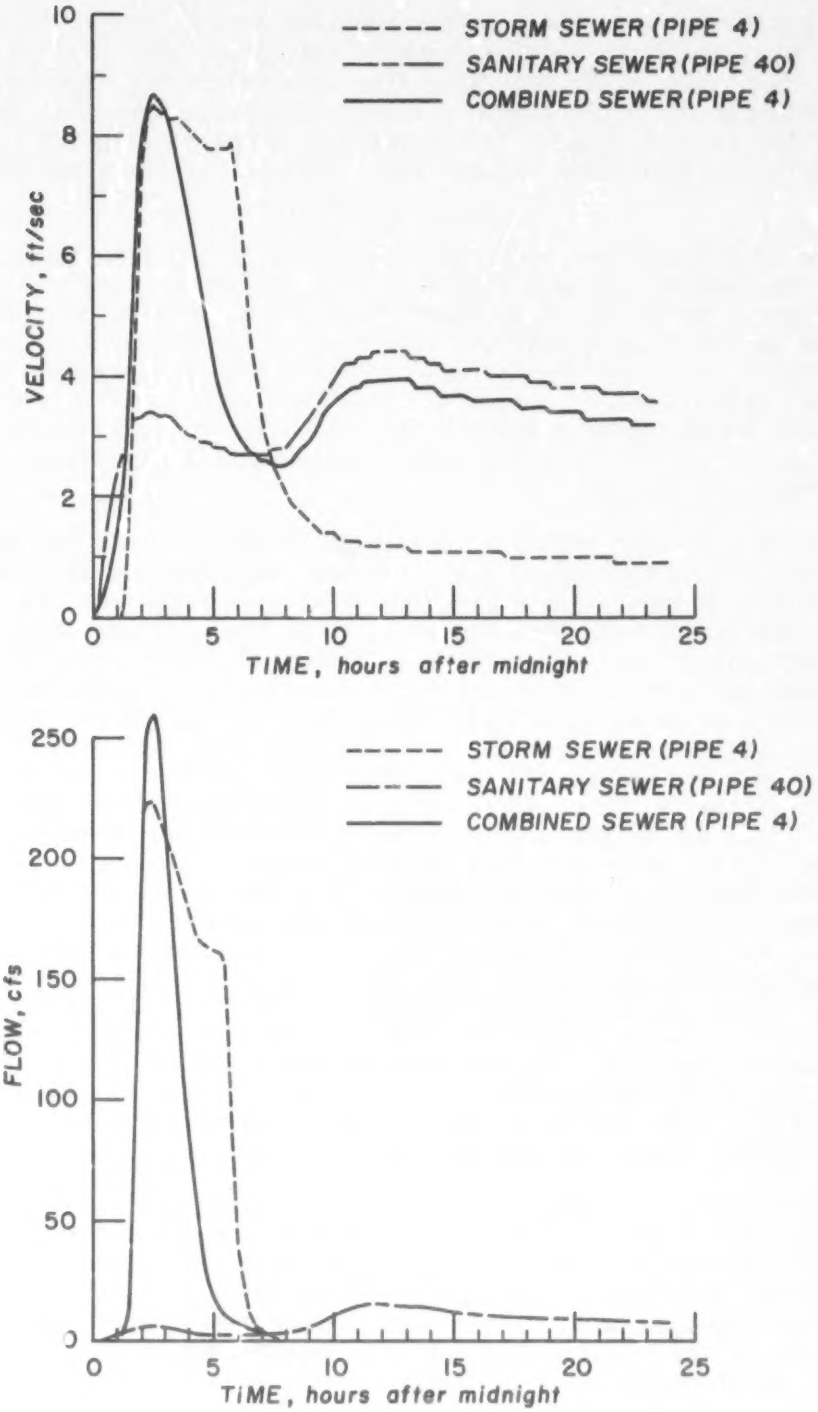


Figure 30
Velocities and Flows in Outfalls from Storm, Sanitary, and Combined Sewer Systems as Simulated

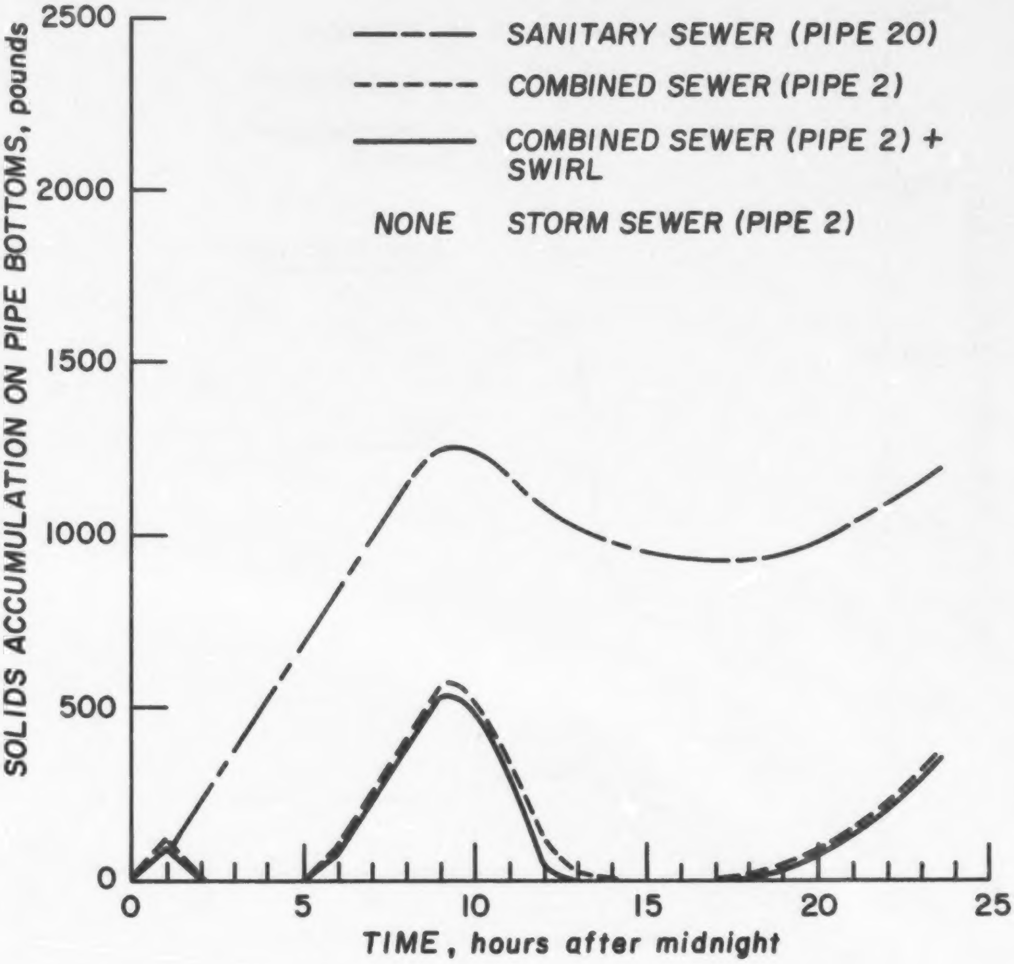


Figure 31
Effect of Sewer System Type on Upstream Accumulation of Solids

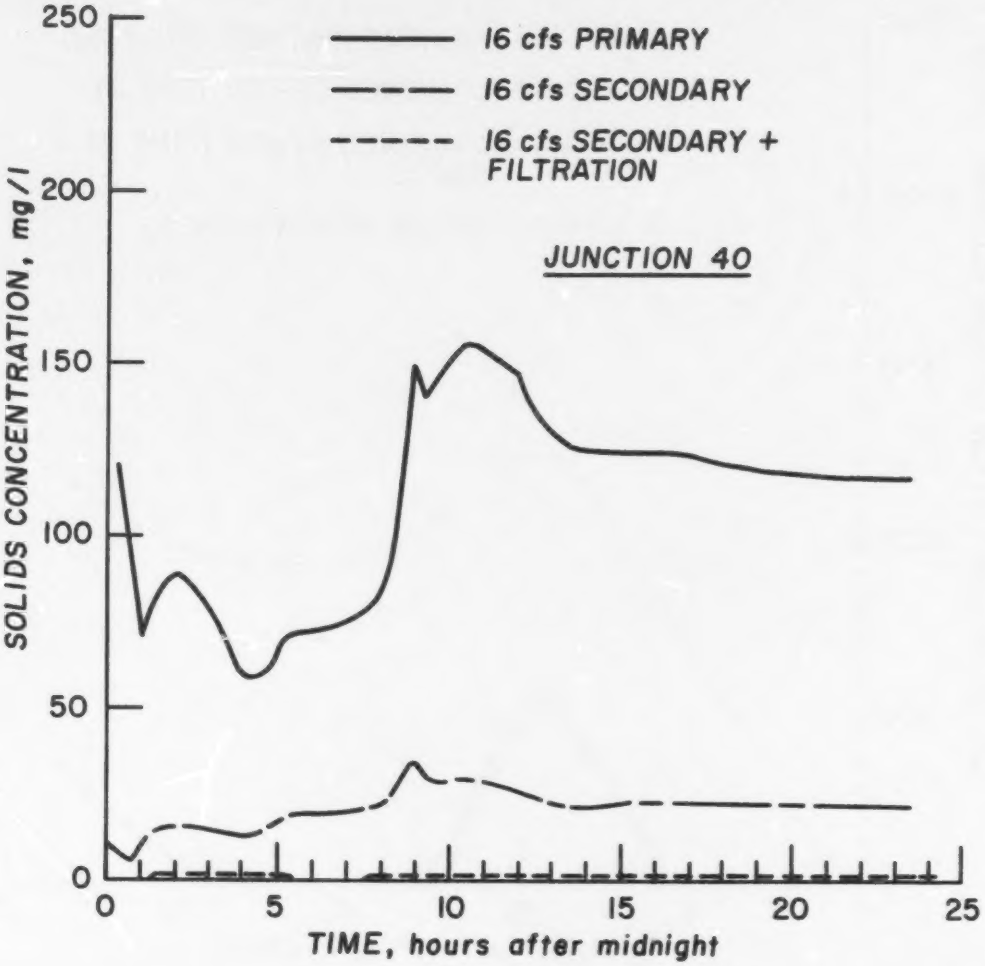


Figure 32
Effect of Treatment Type on Effluent
Solids Concentration from Sanitary Sewer System

sanitary sewer outfall, junction 40. These concentrations may be compared to those for the combined systems in Figure 26. A comparison will show that the sanitary system produced comparable, if slightly higher peak solids concentration in the effluent generally but produced no inordinately high peak during the storm, which of course was not involved in this sewer at all.

Figure 33 shows what are unfortunately not very informative concentration results for the storm sewer outfalls, both with swirl concentrators upstream and without. The results are not very helpful in that they show an enormously high concentration during the runoff event and virtually no concentration thereafter. While this may be partially reflective of reality, it is also true that the enormous peak values occurred in part because the storm sewer outfall was completely dry at the beginning of the storm, and the first drop of water that got to it contained a large amount of added mass from the storm hydrographs which were given their highest solids concentrations at the earliest part of the storm. In other words, the model was a bit overwhelmed in the first time step or two to try to put some mass into very little water, and by the time it had enough water to operate with comfortably, the stormwater concentrations were already becoming very small. Comparison of the combined sewer results in Figure 26, the sanitary sewer results in Figure 32, and the storm sewer results in Figure 33 will show that, at least in our examples, the sanitary sewage had the predominant effect on overall effluent quality. That is not always the case, and more realistic runoff solids concentrations would very likely have produced quite different results.

Finally a comparison of cases similar in slope, shape, and treatment type from the combined sewer set and the separate sewer set is presented in Table 12. The cases, IIIA, IVA, and VIA, were the same as Cases III, IV, and VI in the combined sewer set, except that the treatment plant at pipe 3 was reduced to 16 cfs from 25 cfs or to 32 cfs from 50 cfs to make these cases more nearly comparable to their counterparts in the separate sewer array. The results of the comparison are that the combined sewer systems were always comparable in overall removal capability with the separate sewer systems, and usually they gave rise to a higher degree of removal by treatment and a lower degree of accumulation in the pipes. This cannot be judged to be a general finding, but it is the overwhelming indication for the size, shape, and slope combinations analyzed here.

The Effect on Cost

The costs of all the alternatives analyzed in this project are given in Table 13. Two of the cases in the table, numbers XIII and XIV, are cases in which the surcharged storm sewers in Cases VIII and IX were resized to larger pipes to contain the flow, and the outfalls were made a foot smaller to try to realize some economy. So the pipe sizes in Case XIII were 6, 7, 8, and 7 feet for the four storm sewers, from upstream to down, and the slopes and other characteristics were the same as in Case VIII. The sanitary sewers were unchanged. In Case XIV the storm sewers were also 6, 7, 8, and 7 feet in diameter, and the remainder of the features were the same as in Case IX.

The data in Table 13 are plotted in Figures 34 and 35. Figure 34 is a plot of all the annual treatment costs and the removal percentages attained

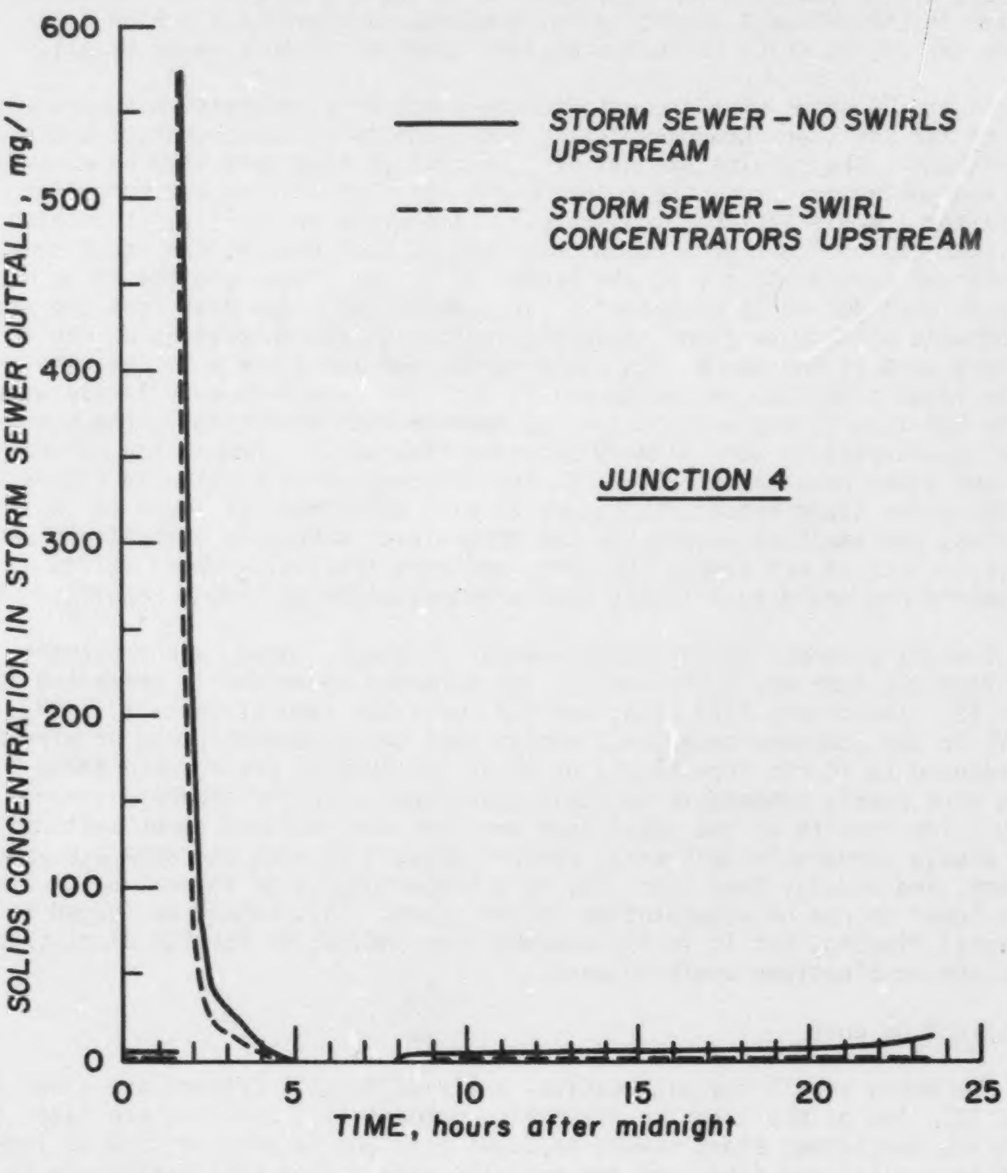


Figure 33
Effect of Upstream Treatment
for Solids Removal in Storm Sewer System as Modeled

TABLE 12. COMPARISON OF RESULTS FOR COMBINED AND SEPARATE SEWER SYSTEMS

		Percentage of Solids			
		Removed by Treatment	Remaining in Pipes	Discharged	Removed Overall
Case I	(25 cfs)**	15.1*	35.1*	49.8	50.2
Case VII	(16 cfs)	11.8	42.7	45.5*	54.5*
Case II	(25 cfs)	25.6*	11.1*	63.3	36.7
Case VIII	(16 cfs)	22.0	17.7	60.3*	39.7*
Case V	(25 cfs)	50.5*	11.1*	38.4*	61.6*
Case XI	(16 cfs)	42.1	17.7	40.2	59.8
Case VI	(50 cfs)	80.7*	7.8*	11.5*	88.5*
Case XII	(16 cfs)	59.6	25.7	14.7	85.3
Case VIA	(32 cfs)	78.7	7.8*	13.5	86.5
Case IIIA	(16 cfs)	20.6	16.9*	62.5	37.5
Case IX	(16 cfs)	21.8*	17.8	60.4*	39.6*
Case IVA	(16 cfs)	53.8*	8.6*	37.6*	62.4*
Case X	(16 cfs)	40.2	17.7	42.1	57.9

* Indicates better finding of the two compared.

**Numbers in parentheses are the treatment capacities of the main dry-weather plants.

TABLE 13. SOLIDS REMOVAL AND COST RESULTS FOR EXAMPLE CASES

Example Case	% Overall Removal	% Removed by Treatment	Annual Pipe Cost, $1,000	Annual Treatment Cost, $1,000	Total Cost $1,000/yr.
I (25)*	50.2	15.1	324.5	494.4	818.9
II (25)	36.7	25.6	312.6	494.1	806.7
III (25)	44.2	20.1	319.4	494.1	813.5
IV (25)	63.1	54.6	312.6	528.7	841.3
V (25)	61.6	50.6	312.6	794.5	1,107.1
VI (50)	88.4	80.7	264.5	918.6	1,183.1
IIIA (16)	37.5	20.6	319.4	382.5	701.9
IV (16)	62.4	53.8	312.6	411.1	723.7
VIA (32)	86.5	78.7	264.5	707.0	971.5
Cunettes-comb. (25)	45.9	20.4	412.6	496.3	908.9
Cunettes-rect. (16)	39.0	19.6	697.7	382.6	1,080.3
VII (16)	54.5	11.8	370.7	376.3	747.0
VIII (16)	39.6	22.0	356.6	375.9	732.5
IX (16)	47.7	16.7	365.0	376.1	741.1
X (16)	57.9	40.2	356.6	408.1	764.7
XI (16)	59.8	42.1	356.6	580.2	936.8
XII (32)	85.3	59.5	314.8	666.4	981.2
XIII (16)	39.6	21.8	388.5	375.9	764.4
XIV (16)	47.8	16.4	398.5	376.1	774.6
Cunettes-sep. (16)	51.1	14.3	471.2	377.2	848.4
Cunettes-enl. (16)	51.0	14.4	527.1	377.4	904.5

*Numbers in parentheses are the capacities of the main dry-weather treatment plants in cfs.

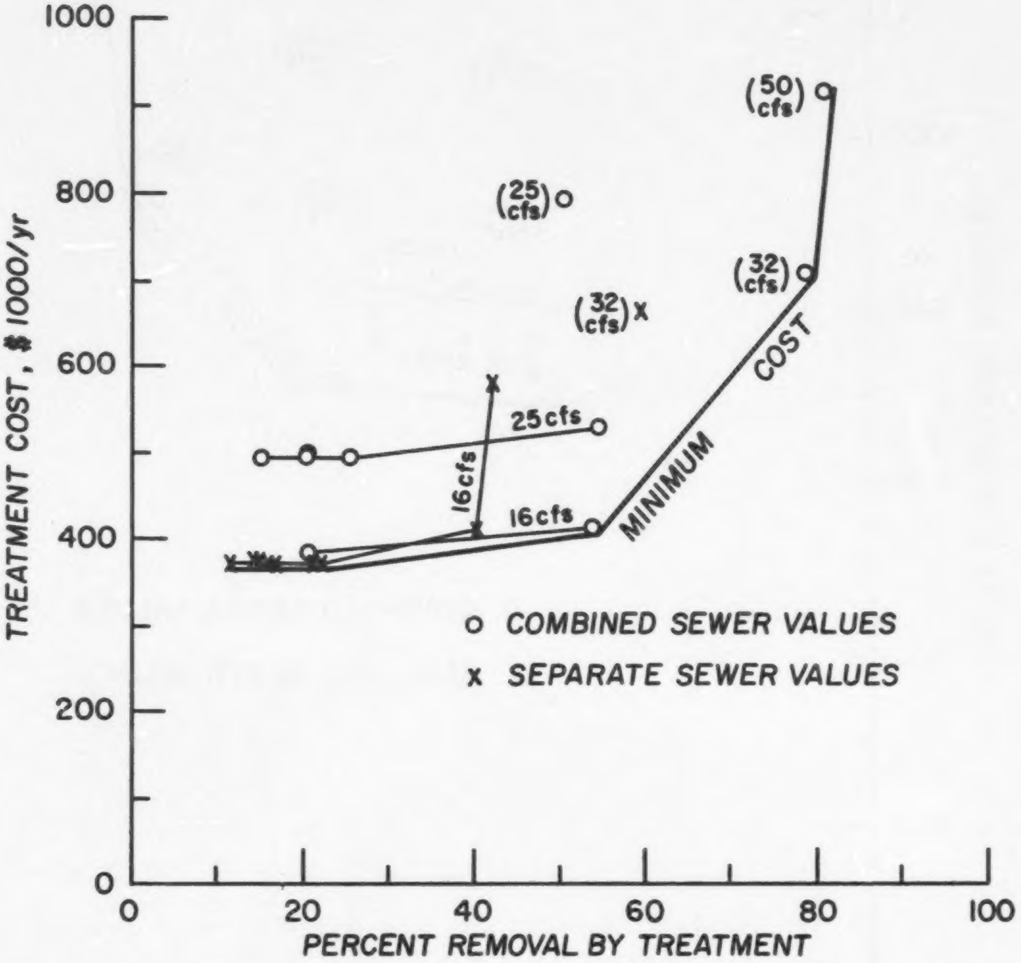


Figure 34
Treatment Costs and Solids Removal Percentages
for Example Cases

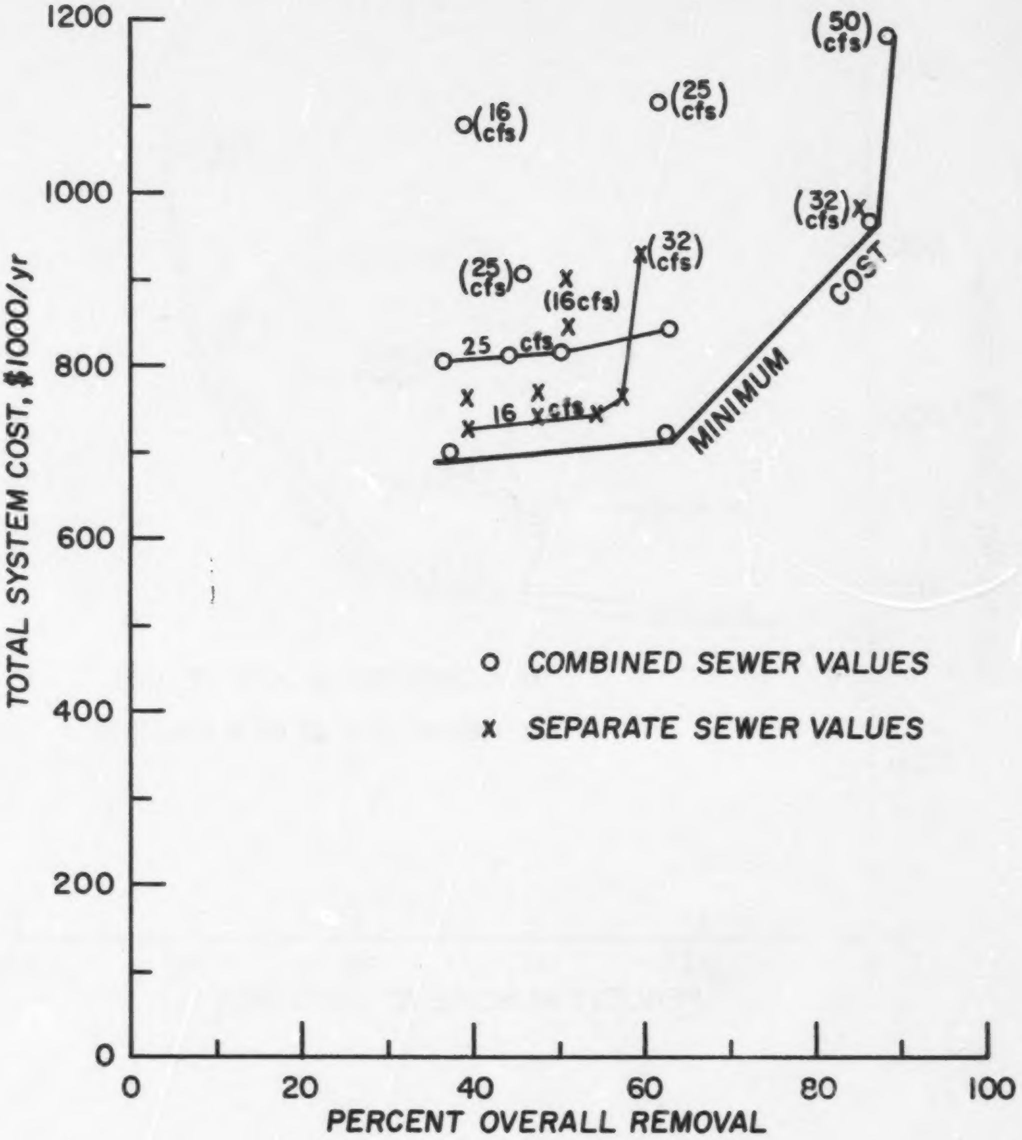


Figure 35
Total Sewer System and Treatment Cost vs.
Overall Removal Percentages for Example Cases

by treatment in the various cases. As might be expected, greater removal percentages resulted from providing more treatment capacity and higher degrees of treatment. At the low end of the removal scale, the separate system with 16 cfs of primary treatment resulted in the minimum cost. For higher solids removal levels, the minimum cost was provided via the combined sewer system with larger and more complete treatment.

Figure 35 shows that the minimum total cost to achieve overall removal was attained throughout the range of removals by the combined sewer system.

These results, like others reported here, are not general findings but are related to the scale and other characteristics of the example problem. In this exercise the combined sewer was the more cost effective because most of the solids were in the sanitary wastes, not in the stormwater. Therefore, it could have been expected that one pipe system to deal with the large majority of the solids would be more cost effective than two pipe systems to deal with all of them. For collection and treatment of more heavily laden storm flows, a separate system of storm and sanitary sewers may prove to be more cost effective. The model developed in this project and described herein can be used as demonstrated to evaluate such a situation as well. Indeed the purpose of the model is to analyze wastewater and stormwater collection and treatment alternatives for a wide variety of conditions.

USE OF THE MODEL IN OTHER APPLICATIONS

Hopefully the example problems discussed here at some length have shown that there is no simple recipe or cookbook approach to complex sewer system analysis. There are many variables to consider and they interact to provide some kinds of benefits while generating opposing kinds of costs. The analysis of a total waste handling system remains an exercise of selective searching. But the examples do suggest some guidelines for sewer analysis that may be helpful. These are itemized below.

1) Sewers that accumulate little or no solids will permit downstream treatment to be most effective, until the treatment capacity is attained. Increasing slope by 50 percent or more in new sewer design can improve treatment by as much as 100 percent, sometimes with a reduction in cost for the pipe system. Therefore, a first thing to try in laying out a new sewer network might be:

   Increase slopes of pipes to the point that accumulation does not occur during the design event as modeled.

2) Although the cunette sections analyzed here were not cost effective, it is possible that the increased cost of installing these special conduits may be offset by improvements in dry-weather treatment efficiency and lower accumulation rates. The second guideline might be:

Analyze sewer cross-section shapes to determine the most cost effective one with a fixed amount and degree of treatment in place.

3) Upstream treatment of stormwater in the example cases was a cost effective addition. Solids removal by treatment increased from 25.6 percent to 54.6 percent for a 4.3 percent increase in cost to provide swirl concentrators in the combined system. A similar benefit was realized with adding swirl concentrators to the storm sewer of the separate system. So the third guideline might be:

   Assign dry-weather treatment to handle the peak dry-weather flow and provide upstream treatment to accommodate all or part of the storm flow. Analyze alternatives to this mixture.

4) Increased treatment capacities and degrees of removal cost money roughly in proportion to the improvement in solids removal attained, until high degrees of removal are achieved. After this point there is a decreasing return in removal for increased treatment efficiency. This must be balanced by trial and error. The fourth guideline, then, would be:

   Analyze alternatives with higher and higher degrees of removal capability until decreasing marginal gains are achieved in solids reduction per treatment dollar spent.

These guidelines have been suggested by the examples analyzed in this project. Quite different situations may well suggest another sequence to follow. But it seems clear that the model described is capable of analyzing the solids behavior and capture in pipe systems of various sizes, shapes, and types. The general finding of greatest significance is probably that accumulation of solids in the pipe system may occur at several points throughout a day and the accumulated solids are likely to be swept past a dry-weather treatment device at precisely the worst moment--when it is already hydraulically overloaded. Design of the collection system is a critical part of design for the treatment plant. Indeed these designs should be conducted simultaneously, and the model developed here permits this.

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Appendix

LISTS OF NEW MODEL SUBROUTINES

S C R D E P

```
      SUBROUTINE SCRDEP
      COMMON/SETTLE/GAMS(10),GAM,PNU,GRAV
      COMMON /SEDMT/ SED(100,10),RED(100,10),AVAIL(10),WL(10),SL(10),
     1               BL(10),TRANS(10)
      COMMON/SOLIDS/SPER(100,10),GAMJI(100,10)
      COMMON/SCOWR/BLCON(100),SLCON(100),WLCON(100),BOT(100),DSOL(10),
     1   TAUC(10),DELTR,NDEPS,ADEP,SD(10),VSS(10),R10,SDW(10),SGW(10),
     2 SDD(10),SGD(10)
      COMMON/JUNCT/ NJ,JUN(100),NCHAN(100,8),MASSIN(100,3),C(100,3),
     1   Y(100,2),YAVE(100),VOL(100),VOLD(100),JSKIP(100)
      COMMON/COND/ NC,NTC,NCUND(100),NKLASS(100),DEEP(100),WIDE(100),
     1   AFULL(100),RFULL(100),LEN(100),NJUNC(100,2),Q(100,2),QAVE(100),
     2   V(100),HYRAD(100),ZP(100,2)
      COMMON/TREET/ REMOV(10,3),NJTRT,NPLAC(20),NTRCLS(20),TOTREM,
     1 QTRT(20),QDES(20),NLOC(100),MASTOT,AMREMV,DEPD
     2 ,ITREAT,IBED
      COMMON/TOTSOL/DEPOS(100),WLT(20),SLT(20),BLT(20),AWASH(100),
     1               SCRSOL(100),DEPOST(100),BDLAST(100),BDFRST(100)
     2              ,WASH(100),TOTWSH,SCRTOT,SCRTIM(100)
      REAL MASSIN,LEN
      REAL MASTOT
C
C**** INITIALIZE VARIABLES
C
      DEPS = NDEPS
      VK = 0.385
C**** UPDATE POUNDS OF SEDIMENT IN POTENTIAL MOTION AT JUNCTIONS

      DO 25 J = 1,NJ
      N = NCHAN(J,2)
      MASTOT = MASTOT + MASSIN(J,2)
      DO  25 ISZ = 1,10
      ALINK = SED(J,ISZ)
      SED(J,ISZ) = SED(J,ISZ) + MASSIN(J.2)*SPER(J,ISZ)
      IF(N.GT.NC) SED(J,ISZ) = SED(J,ISZ) - ALINK
      IF(SED(J,ISZ).LT.0.0) SED(J,ISZ) = 0.0
   25 CONTINUE
C
C     * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
C
C                        MAJOR DO LOOP ON PIPES
C
C     * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
C
      DO 4000 N = 1,NC
      SCRSOL(N) = 0.0
      BDLAST(N) = 0.0
      AWASH(N) = 0.0
      DEPOS(N) = 0.0
      WASH(N) = 0.0
      SCRTIM(N) = 0.0
C
C**** COMPUTE PIPE CONSTANTS
C
      IF (LEN(N).LE.0.0) GO TO 4000
      IF(V(N).LT.0.0.AND.ABS(V(N)).LT.0.0001)V(N) = -0.0001
      IF(ABS(V(N)).LT.0.0001)V(N) = 0.0001
      J1 = NJUNC(N,1)
      J2 = NJUNC(N,2)
      IF (QAVE(N)) 100,200,200
  100 J = J2
      K = J1
      H1 = YAVE(J) + ZP(N,2)
      H2 = YAVE(K) + ZP(N,1)
      GO TO 300
  200 J = J1
      K = J2
      H1 = YAVE(J) + ZP(N,1)
      H2 = YAVE(K) + ZP(N,2)
  300 CONTINUE
      DEP = (YAVE(J1)+YAVE(J2))/2.
      IF(DEP.LE. 0.0) GO TO 4000
      B = QAVE(N)/(V(N)*DEP)
      S = (H1-H2)/LEN(N)
      IF(S.LT.0.000001)S=0.000001
      TAU = GAM*HYRAD(N)*S
      IF(TAU.LT.0.000001)TAU=0.000001
      USTAR = SQRT(TAU*GRAV/GAM)
C
      ISIZ = 0
      DO 400 ISZ = 1,10
      IF(GAMJI(J1,ISZ).GT.62.41.AND.V(N).GE.0.0) ISIZ = ISIZ + 1
      IF(GAMJI(J2,ISZ).GT.62.41.AND.V(N).LT.0.0) ISIZ=ISIZ+1
      WL(ISZ) = 0.0
      SL(ISZ) = 0.0
      BL(ISZ) = 0.0
  400 CONTINUE
      M = ISIZ/2 + 1
      R10 = USTAR*DSOL(M)/PNU
      ADEP = 2.0*DSOL(M)
      IF(ADEP.GE.DEP) ADEP = 0.98*DEP
C *** DETERMINE IF THERE IS TREATMENT AT DOWNSTREAM END OF PIPE
      MT = NLOC(N)
      KT = 1
      RATQ = 0.0
      IF(MT.LT.1) GO TO 402
      KT = NTRCLS(MT)
      RATQ = QTRT(MT)/ABS(QAVE(N))
      WLCON(MT) = WLT(MT)
      SLCON(MT) = SLT(MT)
      BLCON(MT) = BLT(MT)
  402 CONTINUE
      R1 = REMOV(KT,1)*RATQ
      R2 = REMOV(KT,2)*RATQ
      R3 = REMOV(KT,3)*RATQ
      VAB = ABS(V(N))
C
C     COMPUTE HYDRAULIC RADIUS DUE TO GRAINS (RG)
C

      KR = 0
      RG = HYRAD(N)
      FMIN = 0.01
C **   'V/USTAR=5.75LOG(12.27RH'/D65)' - GRAF,EQ 11.77,P. 385
```

```
 1300 F  = V(N) - 5.75*SQRT(S*RG*GRAV)*ALOG10(12.27*RG/DSOL(M))
      FF = ABS(F)
      IF (FF.LT.FMIN) GO TO 1360
      KR = KR + 1
      IF (KR.LE.10) GO TO 1340
      WRITE(6,1320) KR,RG,F
 1320 FORMAT (//,21H ** ERROR MESSAGE ** //
     1         48H COMPUTATIONS FOR HYDRAULIC RADIUS DUE TO GRAINS,
     2         20H FAILED TO CONVERGE.,//26H CONDITIONS AT FAILURE ARE/
     3         14H ITERATIONS  =,I6 / 14H HYD. RADIUS =,F6.2 /
     4         14H F VALUE     =,F6.2 // 23H COMPUTATIONS CONTINUED )
      GO TO 1360
 1340 RT = 0.95*RG
      FT = V(N) - 5.75*SQRT(S*RT*GRAV)*ALOG10(12.27*RT/DSOL(M))
      DRDF = (RG-RT)/(F-FT)
      DR = F*DRDF
      RG = RG - DR
      IF (RG.LT.0.0) RG=RG + 0.5*DR
      IF(RG.LE.0.0) RG = 0.005
      GO TO 1300
 1360 CONTINUE


C     * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
C
C          DO LOOP ON SEDIMENT SIZES
C
C     * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
C
      DO 3000 ISZ=1,10
      IF(SED(J1,ISZ).LT.0.001.AND.V(N).GE.0.0) GO TO 3000
      IF(SED(J2,ISZ).LT.0.001.AND.V(N).LT.0.0) GO TO 3000
C
C     COMPUTE CRITICAL VELOCITY FOR NO DEPOSITION--VC=FL*SQRT(2GD(S-1))
C         'DURAND' EXPRESSION -- GRAF, P. 435 (FL=0.9, SQRT(2G)=8.025)
C
      SG = GAMJI(J,ISZ)/62.4
      VC = 0.9 * 8.025 * SQRT(DEP*(SG-1.0))
C
      TRANS(ISZ) = 0.0
      IF(VOL(J).GT.0.0)TRANS(ISZ)=SED(J,ISZ)*ABS(QAVE(N))*DELTR/VOL(J)
      IF(TRANS(ISZ).GT.SED(J,ISZ)) TRANS(ISZ) = SED(J,ISZ)
      AVAIL(ISZ) = TRANS(ISZ) + BED(N,ISZ)
      W = 0.0
      IF(VAB.LE.0.01) W = VSS(ISZ)
      IF(VAB.GT.0.01.AND.VAB.LT.VC) W = (VC-VAB)*VSS(ISZ)/VC
C
C**** WASHLOAD COMPUTATIONS
C
C     DETERMINE IF THERE WILL BE ANY OR TOTAL SEDIMENTATION

      PART = W*B*LEN(N)/ABS(QAVE(N))

      IF(PART.GT.0.9999) GO TO 1199
      WL(ISZ) = TRANS(ISZ) * (1.0- PART)
      OTHER = TRANS(ISZ) * PART
      SED(J,ISZ) = SED(J,ISZ) - WL(ISZ)
      SED(K,ISZ) = SED(K,ISZ) + WL(ISZ)*(1.-R1)
      AMREMV = AMREMV + WL(ISZ) * R1
      AWASH(N) = AWASH(N) + WL(ISZ)
      WASH(N) = WASH(N) + WL(ISZ)*(1.-R1)
      TOTWSH = TOTWSH + WL(ISZ)
      IF(MT.GT.0)WLT(MT)=WLT(MT)+WL(ISZ)*R1
      SED(J,ISZ) = SED(J,ISZ) - OTHER
      BED(N,ISZ) = BED(N,ISZ) + OTHER
      DEPO = DEPO + OTHER
      DEPOS(N) = DEPOS(N) + OTHER
      DEPOST(N) = DEPOST(N) + OTHER
      GO TO 1200
 1199 BED(N,ISZ) = AVAIL(ISZ)
      DEPO = DEPO + TRANS(ISZ)
      DEPOS(N) = DEPOS(N) + TRANS(ISZ)
      DEPOST(N) = DEPOST(N) + TRANS(ISZ)
      SED(J,ISZ) = SED(J,ISZ) - TRANS(ISZ)
      WL(ISZ) = 0.0
C
C**** BED LOAD COMPUTATIONS
C
 1200 FTCOT = TAUC(ISZ)/TAU
      IF(FTCOT.GT.3.0)FTCOT=3.0
      QOUD = 2./10.**FTCOT
      IF(FTCOT.GT.2.)QOUD=434.31/10.**(2.16838*FTCOT)
      IF(FTCOT.LE.0.3)QOUD=27./10.**(4.77121*FTCOT)
      QS = USTAR*DSOL(ISZ)*QOUD
      GS = QS*GAMJI(J1,ISZ)
      IF(V(N).LT.0.0)GS = QS*GAMJI(J2,ISZ)
C * *  'GS' IS IN UNITS OF 'LBS/SEC-FT'
C
C**** SUSPENDED LOAD COMPUTATIONS
C
      Z= VSS(ISZ)/(VK*USTAR)
      IF(Z.GT.3.0) Z = 3.0
      GSS = 0.0
      DELY = DEP/DEPS
      YY =-0.5*DELY
C
C     COMPUTE REFFRENCE SUSPENDED SEDIMENT CONCENTRATION  (CA)

      CA = GS/(11.6*SQRT(S*RG*GRAV)*ADEP)

C     COMPUTE SUSPENDED SEDIMENT LOAD THROUGHOUT DEPTH

      DO 1500 IY =1,NDEPS
      YY = YY + DELY
      UY = USTAR*(5.5+5.75*ALOG10(USTAR*YY/RNU))
      IF(R10.GT.10.)UY=USTAR*(8.5+5.75*ALOG10(YY/DSOL(M)))
      CONY = CA*(((DEP-YY)/YY)*(ADEP/(DEP-ADEP)))**Z
      GSS = GSS + CONY*UY*DELY
 1500 CONTINUE
C * *  'GSS' IS IN UNITS OF'LBS/SEC-FT'
C
C**** CONDUCT SEDIMENT BALANCE AND TRANSPORT
C
      GS = GS * B * DELTR
      GSS = GSS * B * DELTR
      AVAIL(ISZ) = AVAIL(ISZ) - WL(ISZ)
      GTOT = GS + GSS
      IF(GTOT.LE.AVAIL(ISZ)) GO TO 1600
      GS = GS*AVAIL(ISZ)/GTOT
      GSS = GSS*AVAIL(ISZ)/GTOT
 1600 SL(ISZ) = GSS
      BL(ISZ) = GS
      IF(SL(ISZ).GT.BED(N,ISZ)) GO TO 1650
      SED(K,ISZ) = SED(K,ISZ)+SL(ISZ)*(1.-R2)
      AMREMV = AMREMV + SL(ISZ)*R2
      IF(MT.GT.0)SLT(MT)=SLT(MT)+SL(ISZ)*R2
      BED(N,ISZ) = BED(N,ISZ) - SL(ISZ)
      GO TO 1900

 1650 SED(K,ISZ) = SED(K,ISZ)+BED(N,ISZ)*(1.-R2)
      AMREMV = AMREMV + BED(N,ISZ)*R2
      IF(MT.GT.0)SLT(MT) = SLT(MT)+BED(N,ISZ)*R2
      SL(ISZ) = BED(N,ISZ)
      BL(ISZ) = 0.0
      BED(N,ISZ) = 0.0
      GO TO 2900

 1900 CONTINUE

      IF(BL(ISZ).GT.BED(N,ISZ))GO TO 2600
      SED(K,ISZ) = SED(K,ISZ) + BL(ISZ)*(1.-R3)
      AMREMV = AMREMV + BL(ISZ)*R3
      IF(MT.GT.0)BLT(MT)=BLT(MT)+BL(ISZ)*R3
      BED(N,ISZ) = BED(N,ISZ) - BL(ISZ)
      GO TO 2900

 2600 SED(K,ISZ) = SED(K,ISZ) + BED(N,ISZ)*(1.-R3)
      AMREMV = AMREMV + BED(N,ISZ)*R3
      IF(MT.GT.0)BLT(MT)=BLT(MT)+BED(N,ISZ)*R3
      BL(ISZ) = BED(N,ISZ)
      BED(N,ISZ) = 0.0
C
 2900 SCRSOL(N) = SCRSOL(N) +SL(ISZ)*(1.-R2) + BL(ISZ)*(1.-R3)
      SCRTIM(N) = SCRTIM(N) + SL(ISZ) + BL(ISZ)
      SCRTOT = SCRTOT + SL(ISZ) + BL(ISZ)
      BDLAST(N) = BDLAST(N) + BED(N,ISZ)
C      END OF SEDIMENT SIZE DO LOOP
C
 3000 CONTINUE
C
      IF(MT.LT.1) GO TO 3001
C     'WLCON' , 'SLCON' , AND 'BLCON' ARE THE LBS OF SOLIDS OF 3 TYPES
C     REMOVED BY TREATMENT PLANT 'MT' IN THE CURRENT TIME PERIOD

      WLCON(MT) = WLT(MT) - WLCON(MT)
      SLCON(MT) = SLT(MT) - SLCON(MT)
      BLCON(MT) = BLT(MT) - BLCON(MT)
 3001 CONTINUE
C      END OF PIPE DO LOOP
C
 4000 CONTINUE
```

```
C
C**** COMPUTE CONCENTRATION (LBS/CUBIC FOOT) OF SEDIMENT IN MOTION
C
      DO 5020 J=1,NJ
      C(J,2) = 0.0
      DO 5000 ISZ=1,10
      C(J,2) = C(J,2) + SED(J,ISZ)
 5000 CONTINUE
      C(J,2) = C(J,2)/VOL(J)
      IF(VOL(J).LT.0.01) C(J,2) = 0.0
 5020 CONTINUE
      RETURN
      END
```

SCROUT

```
      SUBROUTINE SCROUT
      COMMON /JUNCT/ NJ,JUN(100),NCHAN(100,8),MASSIN(100,3),C(100,3),
     1   Y(100,2),YAVE(100),VOL(100),VOLD(100),JSKIP(100)
      COMMON /COND/ NC,NTC,NCOND(100),NKLASS(100),DEEP(100),WIDE(100),
     1   AFULL(100),RFULL(100),LEN(100),NJUNC(100,2),Q(100,2),QAVE(100),
     2   V(100),HYRAD(100),ZP(100,2)
      COMMON/SCOWR/BLCON(100),SLCON(100),WLCON(100),BOT(100),DSOL(10),
     1   TAUC(10),DELTR,NDEPS,ADFP,SD(10),VSS(10),R10,SDW(10),SGW(10),
     2 SDD(10),SGD(10)
      COMMON/TREET/ REMOV(10,3),NJTRT,NPLAC(20),NTRCLS(20) ,TOTREM,
     1 QTRT(20),QDES(20),NLOC(100),MASTOT,AMREMV,DEPO
     2 ,ITREAT,IBED
      COMMON/SEDMT/ SED(100,10),BED(100,10),AVAIL(10),WL(10),SL(10),
     1              BL(10),TRANS(10)
      COMMON/TOTSOL/DEPOS(100),WLT(20),SLT(20),BLT(20),AWASH(100),
     1              SCRSOL(100),DFPOST(100),BDLAST(100),BDFRST(100)
     2             ,WASH(100),TOTWSH,SCRTOT,SCRTIM(100)
      COMMON/CNTROL/ TITLE(40),ALPHA(30),BETA(30),NQCYC,NQUAL,DELT,
     1 TZERO,NPRNT,NSTART,INTER,N21,N22,IQCYC,TIME,TE,TEO,TIMEO,T2,T20,
     2   TP,NINREC,N60,ILIST,JN(5),NOUTS,NTROUT
      DIMENSION BS(10)
      REAL MASTOT

      WRITE(6,100) TIME
      WRITE(6,10)
   10 FORMAT(6X,'SUSPENDED SULIDS CONC*S. AT EACH JUNCTION BY SIZE GROUP
     1 (ISZ)---MG/L',/,1X,'JUNC      ISZ=1      ISZ=2      ISZ=3      ISZ=4
     2    ISZ=5      ISZ=6      ISZ=7      ISZ=8      ISZ=9    ISZ=10
     3 TOTAL',/)
      DO 20 J = 1,NJ
      DO 15 ISZ = 1,10
      BS(ISZ) = 0.0
      IF(VOL(J).GT.0.0)BS(ISZ)=(SED(J,ISZ)/VOL(J))/0.00006242
   15 CONTINUE
      CJ = C(J,2)/0.00006242
      WRITE(6,40)J,(BS(ISZ),ISZ=1,10),CJ
   20 CONTINUE
   40 FORMAT(I5,10F10.1,F12.1)
      WRITE(6,11)
   11 FORMAT(1H0,/,12X,'SOLIDS DEPOSITED ON PIPE BOTTOM AT END OF TIME S
     1TEP BY SIZE (ISZ)---POUNDS',//,1X,'PIPE    ISZ=1      ISZ=2      ISZ
     2=3      ISZ=4      ISZ=5      ISZ=6      ISZ=7      ISZ=8      ISZ=9
     3 ISZ=10',/)
      DO 30 N = 1,NTC
      WRITE(6,50)N,(BED(N,ISZ),ISZ=1,10)
   30 CONTINUE
   50 FORMAT(I5,10F10.2)
      WRITE(6,100) TIME
  100 FORMAT(1H1,'TIME =',F8.0,//)

      DO 1000 N = 1,NC
      AA = DEPOS(N) + AWASH(N)
      WRITE(6,101) N,AA
  101 FORMAT('0PIPE NO. =',I8,10X,'LBS ADDED TO PIPE THIS TIME =',F10.1)
      BB = WASH(N) + SCRSOL(N)
      WRITE(6,102) BB
      102 FORMAT(30X,'LBS LEAVING PIPE THIS TIME = ',F10.1)
          MT = NLOC(N)
          IF(MT.GT.0) GO TO 200
          CC = 0.0
          GO TO 201
      200 CC = WLCON(MT)+SLCON(MT)+BLCON(MT)
      201 WRITE(6,103) CC
      103 FORMAT(25X,'LBS REMOVED BY TREATMENT,IF ANY =',F10.1)
          DD = BDLAST(N)
          WRITE(6,104) DD
      104 FORMAT(25X,'LBS REMAINING IN PIPE ON BOTTOM =',F10.1)
          WRITE(6,105) DEPOS(N)
      105 FORMAT(25X,'LBS DEPOSITED THIS TIME,IF ANY = ',F10.1)
          WRITE(6,106) SCRTIM(N)
      106 FORMAT(25X,'LBS SCOURED THIS TIME , IF ANY = ',F10.1)
          EE = DEPOS(N) - SCRTIM(N)
          WRITE(6,107) EE
      107 FORMAT(25X,'NET LBS SCOURED(-) OR DEPOSITED(+)',/
         1       25X,'--NOT INCLUDING WASHLOAD        = ',F10.1)
     1000 CONTINUE

          RETURN
          END
```

SETVEL

```
      SUBROUTINE SETVEL(D,ISZ,W)
      COMMON/SETTLE/GAMS(10),GAM,PNU,GRAV

      GATRY = GAMS(ISZ)
      IF(GATRY.LT.GAM) GATRY = 1.001*GAM
      CON1 = ((4./3.)*D*GRAV*(GATRY-GAM))/GAM
C * *  TURBULENT RANGE
      CD = 0.34
      W = SQRT(CON1/CD)
      R = W*D/PNU
      IF(R.GT.3000.) GO TO 915

C * *  TRANSITION RANGE

      CD = 1.0
      W = SQRT(CON1/CD)
      R = W*D/PNU
      KK = 0
      IF(R.GT.100.) GO TO 5

      CD = 10.0
      W = SQRT(CON1/CD)
      R = W*D/PNU
      KK = 1
      IF(R.GT.2.)GO TO 5

      CD = 200.0
      W = SQRT(CON1/CD)
      R = W*D/PNU
      KK = 2
      IF(R.GT.0.1) GO TO 5
      GO TO 914
    5 K = 0
  900 K = K+1
      R = W*D/PNU
      IF(R.LT.0.1)GO TO 914

C * * ZEROED FUNCTION FOR THE DRAG COEFFICIENT,CD-FAIR AND GEYER,P.586
C  *     F=CON1/W**2.-(24.*PNU/D)/W-(3.*(PNU/D)**.5)/W**.5-0.34

      F = CD-(24.*PNU/D)/W-(3.*(PNU/D)**.5)/W**.5-0.34
      IF(ABS(F).LT.0.005*CD) GO TO 915
      IF(K.GT.1)GO TO 910
      FO = F
      IF(ABS(FO).LT.0.0049)GO TO 915
  910 IF(ABS(F).GT.10.*ABS(FO))GO TO 913

C ***  CORRECTION = -F/(DF/DCD)

      CD1 = CD - F
      IF(CD1.LT.0.34) CD1=0.34
      CD = CD1
      W = SQRT(CON1/CD)

      IF(K.LT.10) GO TO 900
      IF(K.EQ.10) GO TO 913

```

SETVEL

```
      C * *  NONCONVERGENCE ERROR MESSAGE - - PROGRAM TERMINATION

        913 WRITE(6,9130)K,KK,F,FO,CON1,W,D,R
       9130 FORMAT(1H0,10X,'NONCONVERGENCE ERROR MESSAGE',//,'    K','   KK',
           1'    F     ','     FO   ','          CON1  ','          SET VEL',
           2'      DIAM. ','    REY. NUM',//,2I5,2F10.6,2F15.2,F12.5,F12.2)

            STOP

      C * *   STOKES RANGE  (R.LT.0.1)
        914 W = (GRAV/18.)*((GATRY-GAM)/GAM)*(D**2./RNU)
        915 CONTINUE
            RETURN
            END
```

T R E A T

```
      SUBROUTINE TREAT
      COMMON/SCOWR/BLCON(100),SLCON(100),WLCON(100),BOT(100),DSOL(10),
     1   TAUC(10),DELTR,NDEPS,ADEP,SD(10),VSS(10),R10,SDW(10),SGW(10),
     2 SDD(10),SGD(10)
      COMMON/TREET/ REMOV(10,3),NJTRT,NPLAC(20),NTRCLS(20) ,TOTREM,
     1 QTRT(20),QDES(20),NLOC(100),MASTOT,AMREMV,DEPO
     2 ,ITREAT,IBED
      COMMON /JUNCT/ NJ,JUN(100),NCHAN(100,8),MASSIN(100,3),C(100,3),
     1   Y(100,2),YAVE(100),VOL(100),VOLD(100),JSKIP(100)
      COMMON /COND/ NC,NTC,NCOND(100),NKLASS(100),DEEP(100),WIDE(100),
     1   AFULL(100),RFULL(100),LEN(100),NJUNC(100,2),Q(100,2),QAVE(100),
     2   V(100),HYRAD(100),ZP(100,2)
      COMMON/TOTSOL/DEPOS(100),WLT(20),SLT(20),BLT(20),AWASH(100),
     1               SCRSOL(100),DEPOST(100),BDLAST(100),BDFRST(100)
     2              ,WASH(100),TOTWSH,SCRTOT,SCRTIM(100)
      COMMON/COST/ ACC(10),BCC(10),OM1(10),OB1(10),OM2(10),OB2(10),
     1   OM3(10),OB3(10),OM4(10),ICOST,CRF
      REAL MASTOT
      DO 50 M = 1,NJTRT
       N= NPLAC(M)

      K = NTRCLS(M)
      TLREM = WLT(M) + SLT(M) + BLT(M)

      WRITE(6,10) M,N
      WRITE(6,11) QDES(M)
      WRITE(6,12) REMOV(K,1)
      WRITE(6,13) REMOV(K,2)
      WRITE(6,14) REMOV(K,3)
      WRITE(6,15) WLT(M)
      WRITE(6,16) SLT(M)
      WRITE(6,17) BLT(M)
      WRITE(6,18) TLREM

   50 CONTINUE

      WRITE(6,20)
      SBDFRS = 0.0
      SBDLAS = 0.0
      SLEAVE = MASTOT - DEPO - AMREMV + SCRTOT

      DO 100 N = 1,NC
      SBDFRS = SBDFRS + BDFRST(N)
      SBDLAS = SBDLAS + BDLAST(N)
  100 CONTINUE
      CHECK = MASTOT + SBDFRS - SBDLAS - AMREMV
      WRITE(6,21) SBDFRS
      WRITE(6,22) SBDLAS
      WRITE(6,23) MASTOT
      WRITE(6,24) SLEAVE,CHECK
      WRITE(6,25) AMREMV
      AA = DEPO - SCRTOT
      WRITE(6,26) AA
      BB = DEPO - SCRTOT + AMREMV
      WRITE(6,27) BB
      CC = 100.*(MASTOT-SLEAVE)/MASTOT
      WRITE(6,28) CC
```

T R E A T

```

   10 FORMAT(//,3X,'PLANT NUMBER',I3,'  AT END OF PIPE',I3)
   11 FORMAT(10X,'DESIGN FLOW =',F5.1,' FT3/SEC')
   12 FORMAT(10X,'WASHLOAD REMOVAL PERCENTAGE',12X,'=',F10.1)
   13 FORMAT(10X,'SUSPENDED LOAD REMOVAL PERCENTAGE',6X,'=',F10.1)
   14 FORMAT(10X,'BEDLOAD REMOVAL PERCENTAGE',13X,'=',F10.1)
   15 FORMAT(/,10X,'WASHLOAD REMOVED DURING EVENT',10X,'=',F10.0,'LBS')
   16 FORMAT(10X,'SUSPENDED LOAD REMOVED DURING EVENT',4X,'=',F10.0,'LBS
     1')
   17 FORMAT(10X,'BEDLOAD REMOVED DURING EVENT',11X,'=',F10.0,'LBS',/)
   18 FORMAT(10X,'TOTAL LOAD REMOVED DURING EVENT',8X,'=',F10.0,'LBS',/)
   20 FORMAT(1H1,15X,' * * *  OVERALL SYSTEM SUMMARY  * * *',/)
   21 FORMAT(10X,'POUNDS OF SOLIDS ON BOTTOM AT BEGINNING OF EVENT  =',
     1        F10.0)
   22 FORMAT(/,10X,'POUNDS OF SOLIDS ON BOTTOM AT END OF EVENT',8X,'=',
     1       F10.0)
   23 FORMAT(/10X,'POUNDS ENTERING THE SYSTEM DURING THE EVENT',7X,'=',
     1        F10.0)
   24 FORMAT(/,10X,'POUNDS DISCHARGED FROM SYSTEM DURING EVENT',8X,'=',
     1        F10.0,/,39X,'(CHECK VALUE',9X,'=',F10.0,')')
   25 FORMAT(/,10X,'POUNDS REMOVED BY TREATMENT DURING THE EVENT',6X,'='
     1,       F10.0)
   26 FORMAT(/,10X,'NET POUNDS SCOURED(-) OR DEPOSITED(+) IN EVENT',4X,
     1        '=',F10.0)
   27 FORMAT(/,10X,'NET POUNDS REMOVED (DEPOSITION + TREATMENT)',7X,'=',
     1        F10.0)
   29 FORMAT(/,25X,'OVERALL PERCENTAGE REMOVAL',9X,'=',F10.1)
      IF(ICOST.LT.1) GO TO 500
      WRITE(6,101)
      TCC = 0.0
      TDOLY = 0.0
      TCCA = 0.0
      TOCOST = 0.0
      DO 300 M = 1,NJTRT
      K = NTRCLS(M)
      DOL = ACC(K)*QDES(M)**BCC(K)
      IF(K.EQ.6)DOL = DOL+ACC(5)*QDES(M)**BCC(5)
      TCC = TCC + DOL
      DOLY=OM1(K)*QTRT(M)**OB1(K)+OM2(K)*QTRT(M)**OB2(K)
     1    +OM3(K)*QTRT(M)**OB3(K)+OM4(K)
      TDOLY = TDOLY + DOLY
      CCA = DOL * CRF
      TCCA = TCCA + CCA
      TANC = CCA + DOLY
      TOCOST = TOCOST + TANC

      WRITE(6,200)M,K,DOL,CCA,DOLY,TANC
  200 FORMAT(I5,I11,3X,F16.0,F13.0,F12.0,F13.0)
  300 CONTINUE

      WRITE(6,400)TCC,TCCA,TDOLY,TOCOST
  400 FORMAT(/,17X,'TOTALS',F13.0,F13.0,F12.0,F13.0)
  101 FORMAT(1H1,//,20X,'SUMMARY OF TREATMENT COSTS',/,
     1   13X,'(PIPE COSTS FROM TRANSPORT MODEL MUST BE ADDED)',//,
     21X,'TREATMENT  TREATMENT    CONSTRUCTION   ANN. CONS.   ANN. OPER.
     3     TOTAL ANN.',/,1X,'PLANT NO.     TYPE        COST,DOLLARS  COST,
     4$/YR   COST, $/YR    COST, $/YR',//)
  500 CONTINUE
      RETURN
      END
```